CONTROL OF LANDSLIDE BY LOWERING GROUNDWATER LEVEL THROUGH INDUCED PUMPING METHOD

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by

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CERTIFICATION OF APPROVAL

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Approved by,

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TRONOH, PERAK

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CERTIFICATION OF ORIGINALITY

This is to certify that I am responsible for the work submitted in this project, that the original work is my own except as specified in the references and acknowledgements, and that the original work contained herein have not been undertaken or done by unspecified sources or persons.

NUR HANISAH BINTI ISMAIL

ABSTRACT

It is well recognized that groundwater table has important role on the slope failure. Slope failure is one of the serious hazards to the community Landslide tragedies have caused substantial loss of life and damage to property and infrastructure.

The purpose of this project is to examine the soil moisture and its shear strength when ground water table is high due to heavy rainfall. This project also attempts to find a method of controlling groundwater table at the slope as induced by pumping method.

This project is based on a laboratory experimental and slope modeling that would reduce the moisture content of the soil by pumping method. It is expected that the pumping method could control the slope failure.

The experimental work was conducted base on the soil in Bukit Kledang. The soil is silty clay type and has an intermediate value of plasticity index. The parameter for shear strength is also in the range that is appropriate for the soil, which is 5kPa of cohesion and 25° of friction angle.

The analysis from the SLOPE/W software shows that at the existing ground water level, the safety index is lower than 1. Lowering groundwater level improves the stability of the slope as indicated by the improve value of factor of safety (FOS). For the slope height of 10m, the reduction of groundwater level could increase FOS from1.153 to 1.310 however for the slope of greater height such as 100m; the reduction could only increase from 0.659 FOS to 0.971 FOS. Thus, other safety measure is necessary for high slope.

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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND OF STUDY

Landslide is a very serious issue in the world such as in La conchita, California (Hussein, 2007), Hong Kong and also in Malaysia (Chua, 2008). The occurrence of landslide in Hong Kong is high with an average of 300-400 landslides per annum due to the high intensity of rainfall that is 2300mm per year. Another example of landslide occurrence is in Malaysia include the landslides in the year 1993 to 1996, which include the Highland Tower tragedy in December 1993 that took place after continuous of rainfall for 10 days (Chua, 2008), tragedy of landslide in Penang in September 1995, and Pos Dipang landslide in August 1996. At the end of 2008, slope failure became a big issue in Malaysia because of two cases of landslide: one which is on 30th November where a bungalow collapse due to slope failure, and another one on 6th December where a landslide at Bukit Antarabangsa killed 5 people and caused 14 bungalow collapsed. The occurrences of landslide can lead to serious damage of structure, lost of life and high costs, for example in US, the losses amount to \$3.5 billion damage, and claimed 25 to 50 lives annually (Hussein, 2007). All of the case happens when the intensity of the rainfall is high and would definitely increase the instability of slopes. Landslide and heavy rainfall have a close relationship. Slope failure occurs when the moisture content of soil is high that would reduce the strength of the soil. Groundwater level plays an important role in the slope stability and strength of the soil. Therefore, by reducing the groundwater level using pumping method, the risk of the slope failure can be reduced.

1.2 PROBLEM STATEMENT

There are many factors that can lead to landslide. The factors are angle of slope, type of earth material involve, weathering, cut of slope, earthquake, and seepage and high ground water table. Slope saturation by water is the primary cause of landslides. This effect can occur in the form of intense rainfall, snowmelt, changes in ground-water levels, and water-level changes along coastlines, earth dams, and the banks of lakes, reservoirs, canals, and rivers. Therefore, high pore water pressure and high moisture content of the soil, is the main cause landslide to occur.

1.3 OBJECTIVES

The objectives of this study include:

- To study the characteristic of residual soil from granite origin
- To examine the engineering properties of the soil that may become factors of slope failure
- To control moisture content in soil so as to prevent the possibility of slope failure by reducing the groundwater level as induced by pumping method

1.4 SCOPE OF WORK

The scope of work is base on laboratory model under saturated condition as induced by pumping method. Therefore, the findings are limited to the granite origin soil taken at the toe of Bukit Kledang, Ipoh. The soil engineering properties will be examined and subjected to different level of rain water intensity. Analysis on the slope factor of safety will be acarried out by using GEO SLOPE/W software.

CHAPTER 2

LITERATURE REVIEW

2.1 LANDSLIDE

Landslide has occurs in many places through out the world as example in La Conchita, coastal area of southern California. This landslide and earth flow occurred in the spring of 1995. People were evacuated and the houses near to the slide were completely destroyed (Hussein, 2007). The term "landslide" describes a wide variety of processes that result in the downward and outward movement of slope-forming materials including rock, soil, artificial fill, or a combination of these. The materials may move by falling, toppling, sliding, spreading, or flowing. Figure 2.1 shows a graphical illustration of a landslide, with the commonly accepted terminology describing its features.

The various types of landslides can be differentiated by the kinds of material involved and the mode of movement (Varnes, 1987). A classification system based on these parameters is shown in Table 2.1. Other classification systems incorporate additional variables, such as the rate of movement and the water, air, or ice content of the landslide material.

Although landslides are primarily associated with mountainous regions, they can also occur in areas of generally low relief. In low-relief areas, landslides occur as cut-and-fill failures (roadway and building excavations), river bluff failures, lateral spreading landslides, collapse of mine-waste piles (especially coal), and a wide variety of slope failures associated with quarries and open-pit mines. The most common types of landslides are described as follows and are illustrated in Figure 2.2.

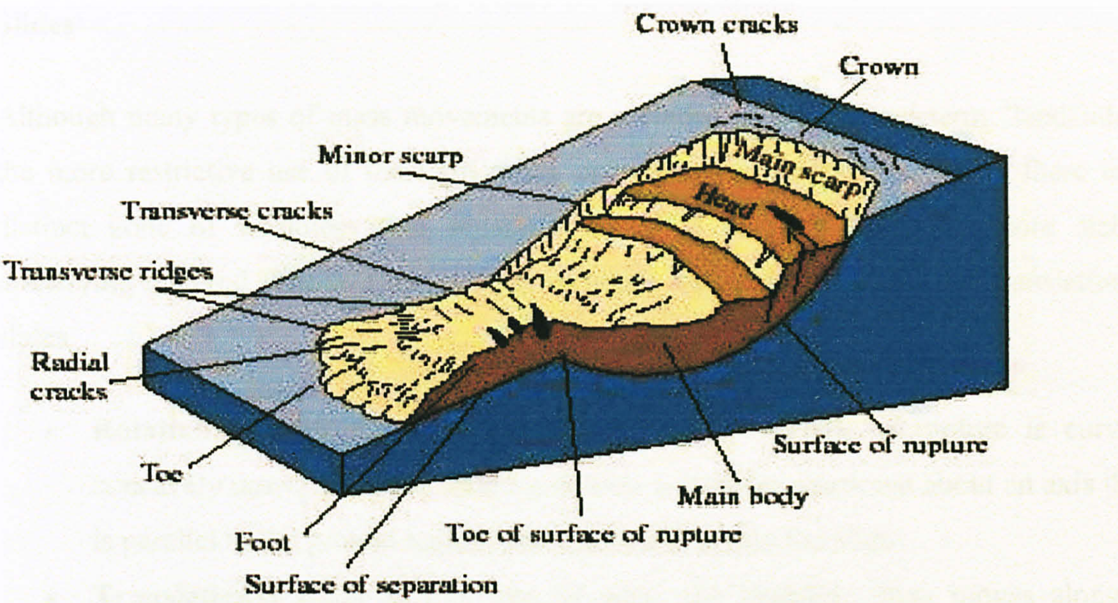


Figure2.1. An idealized slump-earth flow showing commonly used nomenclature for labeling the parts of a landslide. (Source: U.S. Geological Survey)

Table2.1. Types of landslides. Abbreviated version of Varnes' classification of slope movements. (Source: Varnes, 1978)

<table>
<tr><th colspan="2" rowspan="3">TYPE OF MOVEMENT</th><th colspan="3">TYPE OF MATERIAL</th></tr>
<tr><th rowspan="2">BEDROCK</th><th colspan="2">ENGINEERING SOILS</th></tr>
<tr><th>Predominantly Coarse</th><th>Predominantly Fine</th></tr>
<tr><td colspan="2">FALLS</td><td>Rock fall</td><td>Debris fall</td><td>Earth fall</td></tr>
<tr><td colspan="2">TOPPLES</td><td>Rock topple</td><td>Debris topple</td><td>Earth topple</td></tr>
<tr><td rowspan="2">SLIDES</td><td>ROTATIONAL</td><td rowspan="2">Rock slide</td><td rowspan="2">Debris slide</td><td rowspan="2">Earth slide</td></tr>
<tr><td>TRANSLATIONAL</td></tr>
<tr><td colspan="2">LATERAL SPREAD</td><td>Rock spread</td><td>Debris spread</td><td>Earth spread</td></tr>
<tr><td colspan="2" rowspan="2">FLOWS</td><td rowspan="2">Rock flow
(deep creep)</td><td>Debris flow</td><td>Earth flow</td></tr>
<tr><td colspan="2">(soil creep)</td></tr>
<tr><td colspan="2">COMPLEX</td><td colspan="3">Combination of two or more principle type of movement</td></tr>
</table>

2.1.1 Type of Landslide

Slides

Although many types of mass movements are included in the general term "landslide," the more restrictive use of the term refers only to mass movements, where there is a distinct zone of weakness that separates the slide material from the more stable underlying material. The two major types of slides are rotational slides and translational slides.

- **Rotational slide:** This is a slide in which the surface of rupture is curved concavely upward and the slide movement is roughly rotational about an axis that is parallel to the ground surface and transverse across the slide
- **Translational slide:** In this type of slide, the landslide mass moves along a roughly planar surface with little rotation or backward tilting. A block slide is a translational slide in which the moving mass consists of a single unit or a few closely related units that move downslope as a relatively coherent mass

Falls

Falls are abrupt movements of masses of geologic materials, such as rocks and boulders that become detached from steep slopes or cliffs. Separation occurs along discontinuities such as fractures, joints, and bedding planes and movement occurs by free-fall, bouncing, and rolling. Falls are strongly influenced by gravity, mechanical weathering, and the presence of interstitial water.

Topples

Toppling failures are distinguished by the forward rotation of a unit or units about some pivotal point, below or low in the unit, under the actions of gravity and forces exerted by adjacent units or by fluids in cracks

Flows

There are five basic categories of flows that differ from one another in fundamental ways.

Debris Flow

A debris flow is a form of rapid mass movement in which a combination of loose soil, rock, organic matter, air, and water mobilize as slurry that flow downslope (Figure 3F). Debris flows include <50% fines. Debris flows are commonly caused by intense surface-water flow, due to heavy precipitation or rapid snowmelt that erodes and mobilizes loose soil or rock on steep slopes. Debris flows also commonly mobilize from other types of landslides that occur on steep slopes, are nearly saturated, and consist of a large proportion of silt- and sand-sized material. Debris-flow source areas are often associated with steep gullies, and debris-flow deposits are usually indicated by the presence of debris fans at the mouths of gullies. Fires that denude slopes of vegetation intensify the susceptibility of slopes to debris flows.

Debris avalanche

This is a variety of very rapid to extremely rapid debris flow

Earthflow

Earthflows have a characteristic "hourglass" shape. The slope material liquefies and runs out, forming a bowl or depression at the head. The flow itself is elongate and usually occurs in fine-grained materials or clay-bearing rocks on moderate slopes and under saturated conditions. However, dry flows of granular material are also possible.

Mudflow

A mudflow is an earthflow consisting of material that is wet enough to flow rapidly and that contains at least 50 percent sand-, silt-, and clay-sized particles. In some instances, for example in many newspaper reports, mudflows and debris flows are commonly referred to as "mudslides."

Creep

Creep is the imperceptibly slow, steady, downward movement of slope-forming soil or rock. Movement is caused by shear stress sufficient to produce permanent deformation, but too small to produce shear failure. There are generally three types of creep: (1) seasonal, where movement is within the depth of soil affected by seasonal changes in soil moisture and soil temperature; (2) continuous, where shear stress continuously exceeds the strength of the material; and (3) progressive, where slopes are reaching the point of failure as other types of mass movements. Creep is indicated by curved tree trunks, bent fences or retaining walls, tilted poles or fences, and small soil ripples or ridges

Lateral spreads

Lateral spreads are distinctive because they usually occur on very gentle slopes or flat terrain (Figure 2.2F). The dominant mode of movement is lateral extension accompanied by shear or tensile fractures. The failure is caused by liquefaction, the process whereby saturated, loose, cohesionless sediments (usually sands and silts) are transformed from a solid into a liquefied state. Failure is usually triggered by rapid ground motion, such as that experienced during an earthquake, but can also be artificially induced. When coherent material, either bedrock or soil, rests on materials that liquefy, the upper units may undergo fracturing and extension and may then subside, translate, rotate, disintegrate, or liquefy and flow. Lateral spreading in fine-grained materials on shallow slopes is usually progressive. The failure starts suddenly in a small area and spreads rapidly. Often the initial failure is a slump, but in some materials movement occurs for no apparent reason. Combination of two or more of the above types is known as a complex landslide.

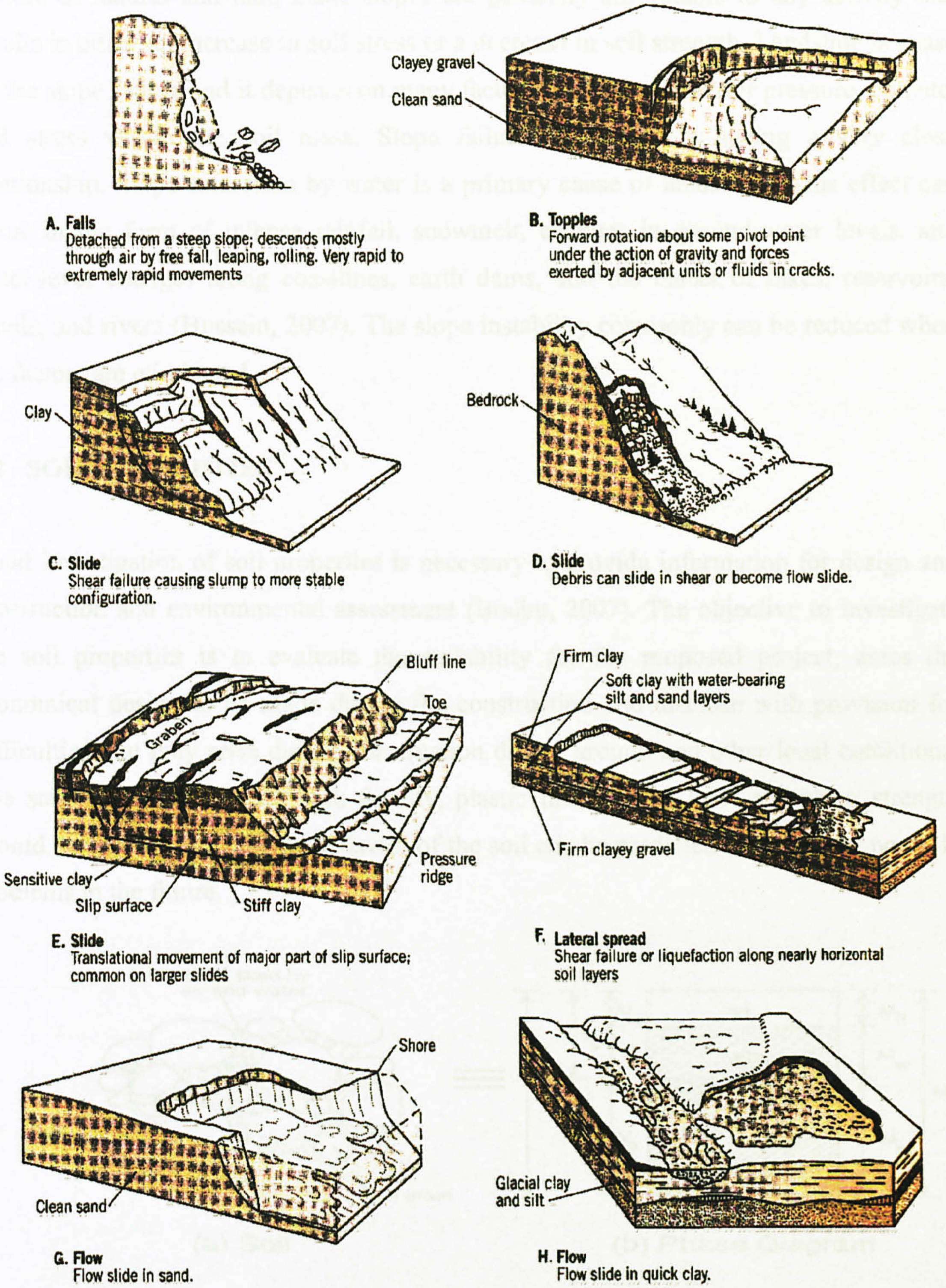


Figure2.2. Example of landslide occurrences (after Varnes, 1978)

2.1.2 Cause of Landslide

Failure of natural and man made slopes are generally attributable to any activity that results in either an increase in soil stress or a decrease in soil strength. Landslide is cause by the slope failure and it depends on many factors such as, pore water pressure, climate, and stress within the soil mass. Slope failure and water is having a very close relationship. Slope saturation by water is a primary cause of landslides. This effect can occur in the form of intense rainfall, snowmelt, changes in ground-water levels, and water-level changes along coastlines, earth dams, and the banks of lakes, reservoirs, canals, and rivers (Hussein, 2007). The slope instability commonly can be reduced when the factors are eliminated.

2.2 SOIL PROPERTIES

Good investigation of soil properties is necessary to provide information for design and construction and environmental assessment (Budhu, 2007). The objective to investigate the soil properties is to evaluate the suitability for the proposed project, asses the economical design to be made during the construction and disclose with provision for difficulties that may arise during construction due to ground and other local conditions. The soil properties such as bulk density, plastic limit, liquid limit and shear strength should be analyzed so that the behavior of the soil can be predicted to encounter possible problems in the future.

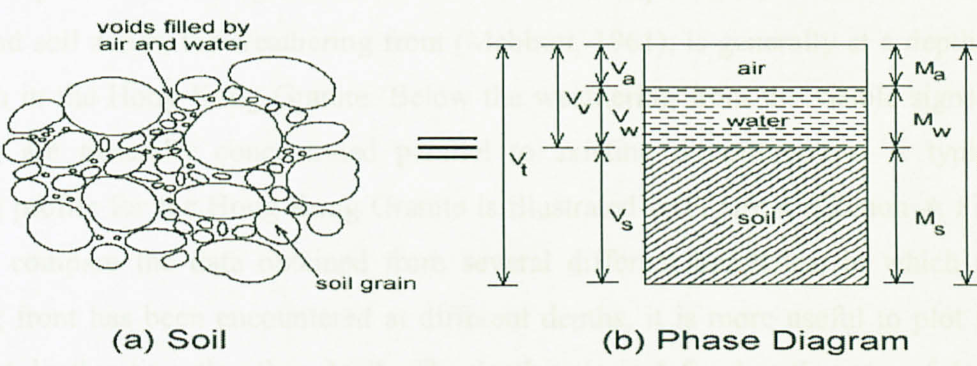


Figure2.3. Phase diagram of soil (Budhu,2007)

Table2.2. General relationship between texture, bulk density and porosity of soils (Source: Juma N.C, 1999)

Textural Class Bulk	Density (Mg/m³)	Porosity (%)
Sand	1.55	42
Sandy Loam	1.40	48
Fine Sandy Loam	1.30	51
Loam	1.20	55
Silt Loam	1.15	56
Clay Loam	1.10	59
Clay	1.05	60
Aggregated Clay	1.00	62

2.3 WEATHERED GRANITE SOIL

The 'completely weathered granite' studied by Howat (1985) represents a transitional stage in the weathering sequence between fresh granite and residual soil. It has been recommended that the term 'completely' be superseded by the term 'extremely' (Dearman 1984), as the weathering processes at this stage is far from complete. Continued weathering results in a collapse of the soil skeleton with a loss of micro fabric and continued discoloration of the material to a characteristic dark reddish brown.

Whilst the depth of weathering can extend to 100 m, the depth of the interface between the rock and soil zones, the weathering front (Mabbutt, 1961), is generally at a depth of 30 to 40 m in the Hong Kong Granite. Below the weathering front the visible signs of weathering are generally concentrated parallel to existing discontinuities. A typical weathering profile for the Hong Kong Granite is illustrated in Figure 4 (Gamon & Finn 1984). To compare the data obtained from several different boreholes, in which the weathering front has been encountered at different depths, it is more useful to plot the data against depth ratio rather than depth. The depth ratio is defined as the ratio of depth of the SPT test or dry density determination to the depth of the weathering front.

The relationship between SPT 'N' value and depth ratio in figure 5 is often distorted by the presence of relict core stones of highly weathered granite or residual soil picking out relict discontinuities within the weathered profile (Gamon 1986). It is considered that the scatter of results is too large to permit further statistical analysis to be meaningful.

The dry density of the granite material decreases with increasing degree of weathering Table 3. The dry density decreases from a maximum of 2.62 mg/m 3 for fresh granite to a minimum of 1.26mg/m- for extremely weathered granite. Dry densities of between 1.40 and 1.71 mg/m 3 have been recorded for the residual soil as a result of densification due to collapse of the soil skeleton. The general trend is for increasing dry density with proximity to the weathering front in figure 6 from approximately 1.5 mg/m 3 at a depth ratio of 0.2 to 1.7mg/m 3 at the weathering front. Material with a dry density between 1.99 mg/m 3 and 2.33 mg/m 3 is difficult to sample using standard drilling techniques, but this friable rock material represents the transitional material between rock and engineering soil (Terzaghi & Peck 1967).

The weathering grades presented in Table 3 are based on a balance of field identification criteria such as material strength, colour, the 'gritty' or 'clayey' nature of feldspars, slakeability, presence or absence of relict texture and macro fabric, and the presence or absence of a discolored rind adjacent to a discontinuity surface (Gamon & Finn 1984). The assessment of weathering grades using this scheme is subjective, which may account for the wide range of dry densities recorded, and a more objective scheme using index properties such as particle size distribution characteristics and dry density should be used to classify the weathering grades for engineering purposes (Gamon, 1986). As granite weathers from fresh rock to residual soil there is an increase in the micro fracture intensity (Baynes & Dearman 1978), a reduction in the strength of the feldspars (Matsuo & Nishida 1968) and a conversion of feldspars to clay minerals.

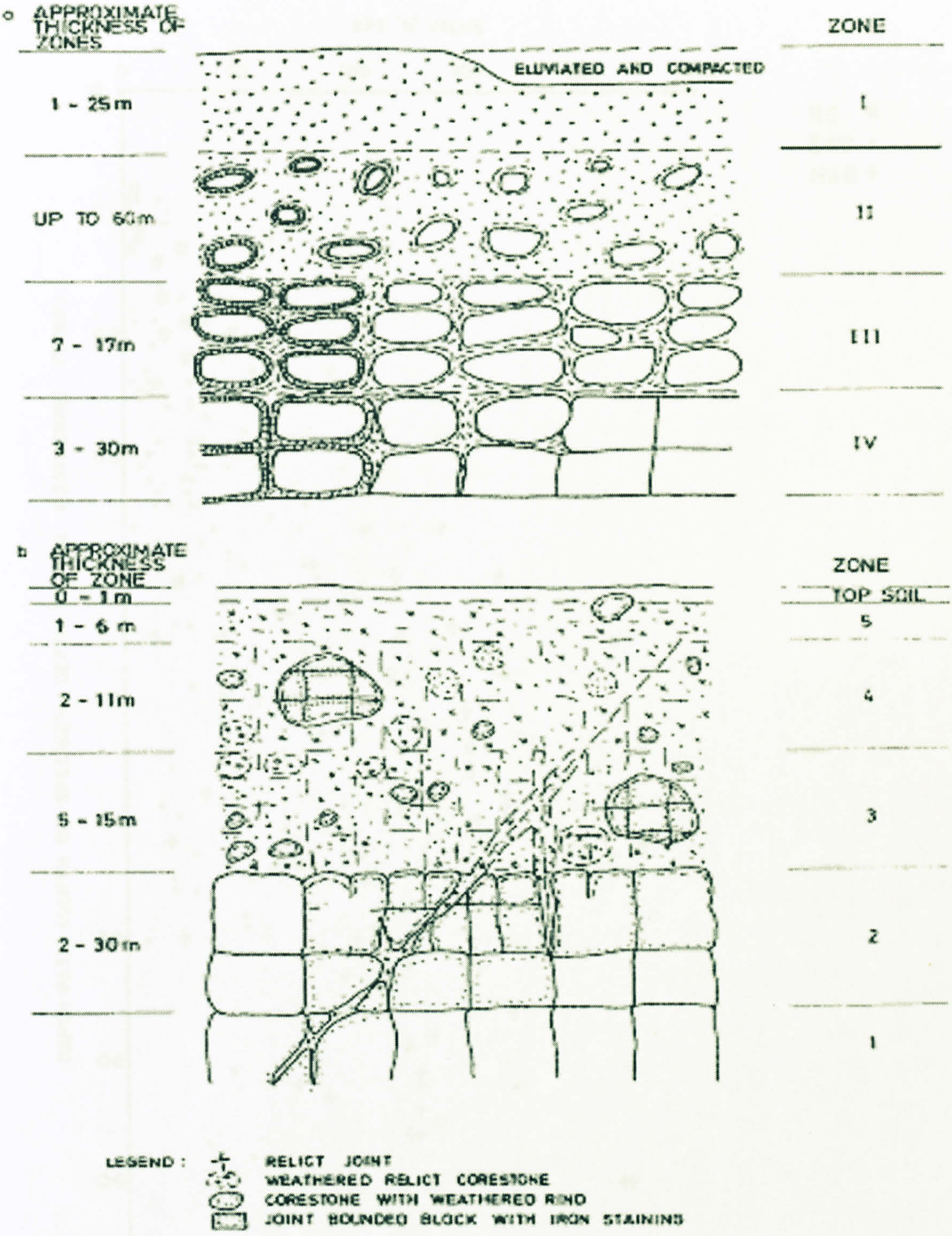


Figure2.4. Zones in weathered granite in Hong Kong.

(a) Zones of a mature profile of weathering on granite (after Ruxton & Berry 1957);

(b) a typical weathering profile for Hong Kong granite

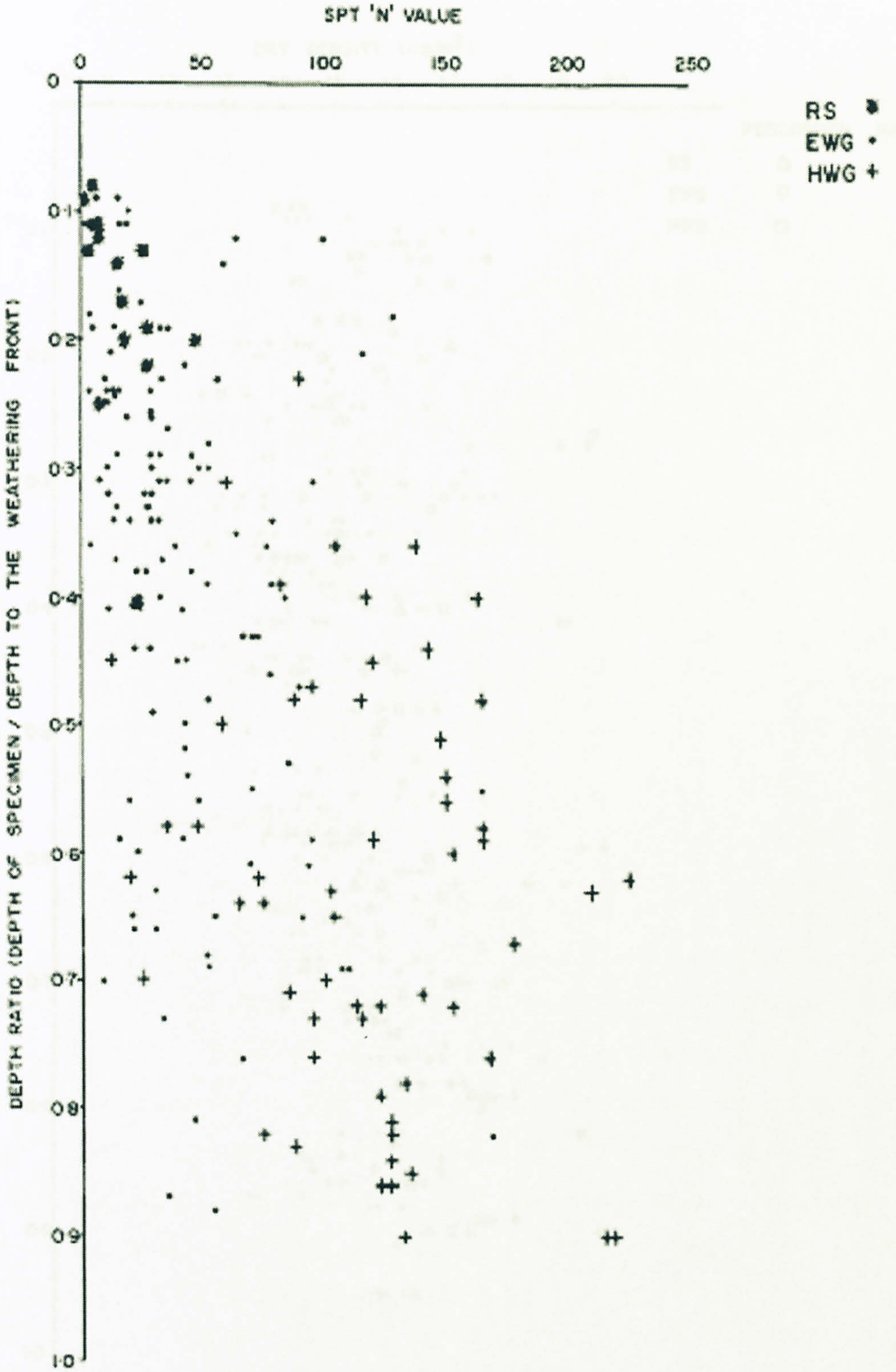


Figure2.5. The relationship between SPT 'N' value and depth ratio

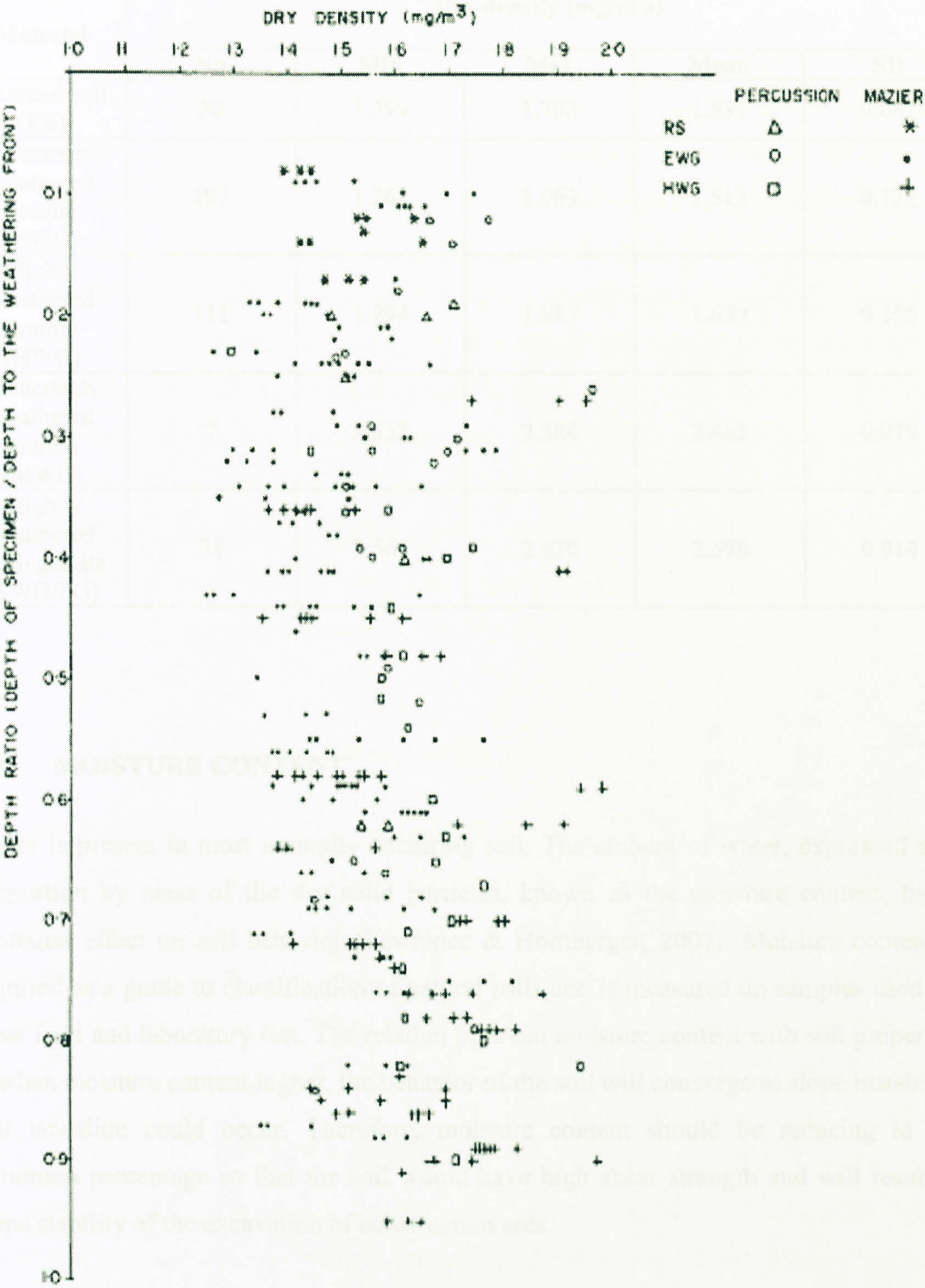


Figure2.6. The relationship between dry density and depth ratio.

Table2.3. Statistical summary of dry density determinations (Gamon 1986)

Material	Dry density (mg/m 3)				
	No	Min	Max	Mean	SD
Residual soil (RS)	20	1.399	1.708	1.531	0.087
Extremely weathered granite (EWG)	197	1.262	1.963	1.513	0.122
Highly weathered granite (HWG)	111	1.294	1.987	1.639	0.156
Moderately weathered granite (MWG)	7	2.332	2.584	2.485	0.079
Slightly weathered fresh granite (SWG/FG)	31	2.563	2.620	2.598	0.014

2.4 MOISTURE CONTENT

Water is present in most naturally occurring soil. The amount of water, expressed as a proportion by mass of the dry solid particles, known as the moisture content, has a profound effect on soil behavior (Lawrence & Hornberger, 2007). Moisture content is required as a guide to classification of natural soils and is measured on samples used for most field and laboratory test. The relation between moisture content with soil properties is when moisture content higher, the behavior of the soil will converge to slope instability and landslide could occur. Therefore, moisture content should be reducing to the minimum percentage so that the soil would have high shear strength and will result to slope stability of the excavation of construction area.

2.5 GROUNDWATER

In saturated groundwater aquifers, all available pore spaces are filled with water (volumetric water content = porosity). Above a capillary fringe, pore spaces have air in them too. Most soils have water content less than porosity, which is the definition of unsaturated conditions, and they make up the subject of vadose zone hydrogeology. The capillary fringe of the water table is the dividing line between saturated and unsaturated conditions. Water content in the capillary fringe decreases with increasing distance above the phreatic surface. One of the main complications which arise in studying the vadose zone is the fact that the unsaturated hydraulic conductivity is a function of the water content of the material. As a material dries out, the connected wet pathways through the media become smaller, the hydraulic conductivity decreasing with lower water content in a very non-linear fashion. A water retention curve is the relationship between volumetric water content and the water potential of the porous medium. It is characteristic for different types of porous medium. Due to hysteresis, different wetting and drying curves may be distinguished.

2.5.1 Effect of Rainfall on Groundwater Level

Groundwater level in clay slopes generally rise during the wet season and rise further during and immediately after storms. With the advent of automatic data acquisition systems, it is possible to monitor piezometer levels continuously. This provides a better understanding of the lag between rainfall and groundwater response, the nature of the upward spikes during high intensity rainfalls, the general level of seasonal rise during the wet season.

2.5.2 Groundwater Blowout Landslides

Groundwater blowout landslides occur where a relatively permeable soil overlies a less permeable soil, resulting in perched groundwater and seepage towards the slope face. The high groundwater levels and seepage towards the slope face result in destabilizing seepage pressure and reduced soil strength. Seeps and springs that form where groundwater exits the slope face often because erosion that can undermine and overstepped a slope further reducing the stability.

Figure 2-2 (sheet 1 and sheet 2) in appendix shows four simplified sketches of a groundwater blowout landslide, together with several alternatives for reducing the likelihood of a landslide. Because the primary driving force is groundwater seepage, suitable remedial measures usually include drainage to lower the groundwater level and to control seepage at the slope face. Drainage measures usually are most effective when they intercept groundwater at the contact between the relatively permeable soil and the underlying less permeable soil.

Sketch A on Figure 2-2 (Sheet 1) in appendix shows the application of an interceptor trench subdrain and a springhead drain. Both improve stability by lowering the groundwater level in a landslide or potentially unstable slope, thereby reducing the driving forces and increasing the soil strength. The springhead drain is used to collect water that emerges from the slope in a concentrated area, thereby reducing erosion potential and improving stability. Trench subdrains generally are applicable to slopes where the contact with the underlying low permeability material is relatively shallow. An interceptor trench subdrain is installed across the slope to intercept the groundwater before it reaches the slope face. Sketch B in appendix shows another type of trench subdrain, called a finger drain. It is similar in construction to an interceptor trench subdrain, except that it is installed along the slope fall line (perpendicular to slope contours).

Sketch C on Figure 2-2 (Sheet 2) in appendix shows two alternatives for drilled drains: horizontal drains and directionally drilled drains. Drilled drains are typically used to improve stability of slopes and landslides where the groundwater cannot be intercepted with trench subdrains, or where it is not practical to excavate trench subdrains. Drilled drains are commonly used to improve the stability of large deep-seated landslides. Horizontal drains are drilled from the slope face, which limits their application to sites that have suitable access near the toe of the landslide mass. Directionally drilled drains usually are installed from the top of the slope and can be aimed to intercept a specific zone where the drainage is needed. Vertical wells (not shown) can be used in special cases; however, their suitable application is limited. Vertical wells require continual pumping to maintain lower groundwater levels. As such, they incur the cost of electricity and are subject to power outages during critical rainy periods.

A replacement earth buttress is sometimes used to improve a marginally stable slope and more commonly to repair a landslide that has already occurred. As shown by Sketch D in appendix, the landslide mass or potentially unstable soil is removed and replaced with a well drained fill material. In some cases, the excavated soil can be recompacted to form the earth buttress, while in others a suitable imported backfill is compacted to form the earth buttress. In either case, an effective drainage layer and subdrain should be constructed under the earth buttress

2.5.3 Analytical Estimation of Groundwater Rise

Lambe(1962) have derived a formula to calculate the rise of Z of the 'wetting front' (Figure2.7) during a storm:

$$Z = \frac{kt}{n\left(S_f - S_o\right)} \qquad \text{Eq. (1)}$$

where,

k = saturated coefficient of permeability;

t = elapsed time of rainfall;

n = soil porosity;

S_o, S_f = initial and final degrees of saturation.

This equation has been put into practice in Hong Kong where the height of the wetting front is added to the winter groundwater level to estimate the peak groundwater levels. For this purpose, a rainfall return period of 10 years is commonly used. Typically, the wetting period band thickness is calculated to be about 2m (6.6 feet)

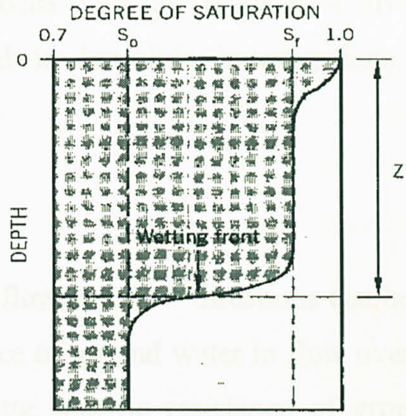


Figure2.7. Advance of the wetting front due to rain infiltrating into soil (after Lambe, 1962)

2.5.4 Ground Water Investigation

Investigation of ground water, which is a driving force of sliding, includes determining ground water level, pore water pressure, ground water logging, ground water tracing test, pumping test, water quality analysis, electricity survey, geothermal survey, and geophysical logging (electric logging and radioactive logging). Based on the results of the above measurements and tests, ground water control works can be planned and designed.

Ground Water Level Observation

As a general rule, ground water levels should be measured in all the boreholes. In some of the more important boreholes, continuous rainfall data will be kept by an automatic recorder to determine the correlation between the slide movement and rainfall and ground water level, and will collect data on the ground water distribution and movement regime.

Pore Water Pressure

Ground water levels in boreholes will often reflect seepage from highly fractured formations or indicate the water level of a predominant aquifer. Therefore, for stability analysis, it is best to measure pore water pressure along the slide plane. Sometimes it is difficult to accurately estimate the depth of the slide plane. In such cases it is desirable to install piezometers in the beds with low seepage or low shear strength. The standard piezometers that are used in landslide investigations must be durable and open piezometer water level type.

Ground Water Logging

Locations of ground water flow and flow directions can be determined by measuring the increasing specific resistance of ground water in flow over time. The measurements will be continued often lowering specific resistance of ground water by injecting a salt solution into the borehole. There should be at least two borings for ground water logging at the head portion of the landslide where abundant ground water is expected. The measurement results should be recorded along with the boring logs, and the relationship between the location of ground water flow and bed, and magnitude and variation of specific resistance of ground water would show the overall ground water flow

Ground water Tracer Tests

Tracers such as a soluble dye, or inorganic chemicals (NaCl) are injected into a borehole. Water samples are then collected chronologically from springs, other boreholes, wells and ponds within or outside the landslide, and are analyzed for the tracer to estimate the ground water flow direction(s) and permeability. This data is used for basic information for the design of dewatering works.

Drawdown Test

In order to estimate the yield and to calculate the coefficient of permeability, water within a borehole is pumped to certain levels after rising the boring casing every 2 to 3m. A time-recovery curve can then be plotted using Jacob's and other formulas, and the coefficient of permeability can be determined.

Geothermal Investigation

This procedure utilizes ground temperature measurements throughout the study area, including ground temperatures near the ground water veins. By measuring the temperature differences at non-ground water areas and near ground water veins, it is possible to isolate the ground water veins where the temperature difference between the two is large. By conducting the geothermal investigation in summer months or winter months where near surface ground temperature is influenced by air temperature, good results have been obtained for the isolation of relatively shallow ground water.

2.6 IMPROVEMENT METHOD

Intercepting groundwater upslope and from within the slope can reduce landslide potential and improve stability of existing landslides (Cruden and Varnes, 1996). Groundwater improvements can be effective on many slopes, and when used appropriately, they are often the most cost-effective approach. The primary goal is to remove groundwater in areas where groundwater reduces stability by adding weight to potentially unstable soils, causes seepage forces, and reduces the soil strength. However, capturing water flowing within the ground requires some different methods compared to surface water improvements. Common groundwater improvement methods include:

- Interceptor trench subdrains and finger drains
- Springhead drains
- Drainage blankets
- Drilled drains

All groundwater improvement schemes should be designed based on the site-specific subsurface conditions. To perform effectively, the system must lower the groundwater level near the landslide failure surface, which requires an understanding of the soil and groundwater conditions that cause instability at the location. The following sections provide a description of common groundwater improvements with some typical design details and requirements.

2.6.1 Interceptor Trench Subdrains and Finger Drains

Trench subdrains are relatively narrow trenches that contain a drainage pipe and permeable backfill. Figure 2-7 (sheet 1, sheet 2) in appendix shows typical trench subdrain cross-sections. Groundwater preferentially flows into the permeable trench backfill and then into the drainage pipe at the bottom of the trench. From there, it is conveyed into a tightline that discharges the groundwater to a suitable discharge location. Trench subdrains are most effective when they penetrate at least 1foot below the contact between the layer being drained and underlying clay, silt, or less permeable layer. This contact is commonly also at or close to the slide plane. Two basic types of trench subdrains include interceptor trench subdrains and finger drains. An interceptor trench subdrain is usually oriented across the slope (parallel to the contours) to intercept groundwater as it flows downslope. Finger drains are frequently used to lower the water level within an active landslide mass by extending a trench subdrain from the toe of the slope up into the landslide debris. Other than their orientation (perpendicular to the contours), finger drains are constructed in the same manner as trench subdrains.

Trench Excavation

Most trench subdrains are excavated using a backhoe or a track-mounted excavator. Therefore, the practical depth for most trench subdrains is about 15 feet or less. Track-mounted excavators are available that can excavate 20 feet deep or more. However, deep trenches are often difficult and expensive to excavate because of the shoring required to maintain stable trench sideslopes. Where groundwater is shallow and site access is limited, hand dug trench subdrains may be practical.

The depth of the trench is generally determined by the maximum practicable depth of the excavating equipment, site conditions, shoring requirements, and other project limitations. As mentioned previously, trench subdrains are most effective when they penetrate though the layer being drained and at least 1 foot into an underlying less permeable soil.

Excavating open trenches in marginally stable soil is often difficult because of groundwater infiltration and the tendency for the trench sidewalls to collapse. Where practical, we recommend beginning the excavation at the outfall and proceeding upslope to allow water to drain away from the advancing trench excavation. It may be necessary to periodically stop work for a day or more to let the site drain before advancing the trench.

Drainage Pipe

The drainage pipe in an interceptor trench subdrain typically consists of a 6-inch (minimum) diameter slotted or perforated plastic pipe. The slots or perforations in the drainage pipe allow water to enter the pipe. However, the drainage pipe only conveys water when the groundwater level rises higher than the pipe invert. When lower water levels are present in portions of a trench subdrain system, the water flows through the surrounding permeable trench backfill. Therefore, the pipe may not need to be placed at precise grades. The pipe should be graded to drain continuously with no sags or depressions where water could infiltrate into the subgrade.

The drainage pipe at the bottom of the trench may consist of rigid or flexible and perforated or slotted pipe. Each type has its advantages and disad4vantages. The rigid pipe is more durable and less susceptible to crushing during installation. However, a worker must be present in the trench to fit the pieces of pipe together and to prepare the bedding gravel. Having workers in a trench generally requires shoring, which results in a higher cost and a longer time for construction. The other alternative, flexible plastic pipe, is easily crushed if workers are not careful or if it is not properly bedded. The primary advantage of the flexible pipe is that it can be lowered down into a deep, unshored trench excavation without workers being in the trench.

The size of the perforations or slots should be compatible with the drainage backfill material around the pipe and the anticipated groundwater flow rates into the pipe. The following paragraph describes filter requirements for perforations and slots in more detail.

Trench Backfill

The trench backfill material depends on the anticipated groundwater inflow and the grain size of the surrounding soil. The backfill should be sufficiently permeable so that water easily flows from the surrounding soil into the trench subdrain. However, the backfill should also act as a filter to prevent migration of the surrounding soil into the subdrain trench. This migration process, known as piping, can eventually plug the subdrain pipe, the drainage backfill or both. In some cases, extensive piping can cause extensive settlement around the trench subdrain from the ground lost by piping. The backfill must also be compatible with the perforated or slotted drainage pipe. If the openings are too small, water cannot enter the pipe fast enough. However, if the openings are too large, the backfill will enter and plug the pipe. To meet the requirements of adequate permeability and filter characteristics, the backfill material should be designed for each specific situation. The following paragraphs describe examples of typical backfill materials that have been successfully used in Seattle soils.

In the example shown on Figure 2-7, Sheet 1 of 2, in appendix the perforated or slotted pipe at the bottom of the trench is bedded in washed pea gravel to provide a highly permeable material directly around the pipe. The pea gravel should be underlain by a geotextile where it rests on silt or clay soil. The remaining backfill can be less permeable, because the groundwater is flowing across a larger area. Therefore, a clean drainage sand or sand and gravel often is appropriate, as shown on the figure. Modified City Type 26 aggregate (Seattle Standard Specifications, 1989, Section 9-03.16) provides adequate permeability for many applications. It is also an adequate filter material for many Seattle soils. The example shown on Figure 2-7, Sheet 2 of 2, in appendix shows a slotted pipe with drainage sand and gravel used for both bedding and backfill.

Backfill is placed in the trench in layers and either tamped with a backhoe or systematically compacted. Generally, if the trench subdrain is located in a landscape area or an unused portion of property, the backfill can be moderately tamped in place with the backhoe bucket to reduce subsequent settlement. However, backfill in trench subdrains located where subsequent settlement of the backfill is not appropriate should be placed and compacted as structural fill material.

Tight line Connections

At the end of the trench subdrain and/or prior to daylighting the drainage pipe on the slope, the slotted or perforated pipe should connect to a tightline pipe. At this transition, the subdrain trench should be filled with concrete or clay to force water from the permeable subdrain trench backfill into the slotted or perforated pipe. Figure 2-8 refer in appendix shows an example of a drainage dam constructed with concrete or compacted clay. From the concrete or clay dam, the tight line should extend to a suitable discharge location.

Trench Cover

The upper 12 to 18 inches of the trench subdrain should be backfilled with a relatively low permeability material to prevent direct infiltration of surface water. Often the soil excavated from the trench is adequate because it should have a similar or lower permeability than the surrounding soil when recompacted in the trench. However, if the trench backfill must be compacted as structural fill, the trench excavation spoils may be too wet to achieve sufficient compaction without some drying and aeration.

In non-structural areas, where compaction is moderate, the backfill should be mounded slightly over the trench to prevent low areas from forming when the trench backfill settles. The surface should be graded to prevent water from ponding near the trench subdrain.

A paved or lined swale installed in conjunction with an interceptor trench subdrain can be used to limit infiltration into the trench subdrain. This remediation tactic is commonly used to control both surface and groundwater near the crest of a slope or close to the edge of a bluff, as shown on Figure 2-1, Sheets 1 and 2 of 3 in appendix.

Geosynthetic Applications

Geosynthetic materials have been used in several trench subdrain applications. These include geotextiles that provide a filter for trench backfill materials and composite drainage materials that form both the drainage material and filter material.

Geotextiles can be used to separate the permeable trench backfill from the surrounding soil and prevent migration of fines into the trench subdrain. For this alternative, the drainage backfill could be a coarse-grained permeable soil that is a poor filter for the surrounding soil, such as uniformly graded gravel. The geotextile would be selected based on its ability to pass water and its filter characteristics to prevent migration of fines from the surrounding soil. The geotechnical engineer should specify this application on a case-by-case basis after careful consideration of the soil conditions. Note that geotextiles are made for many purposes. Therefore, not all geotextiles are appropriate for this application. In deep trenches where shoring boxes are required, the use of a filter geotextile can increase the time and labor costs of the project. Refer to Figure 2-7, Sheet 1 of 2, in appendix for the use of a geotextile to separate pea gravel from on-site soil.

One common misuse of geotextiles is wrapping the fabric directly around the perforated subdrain pipe. Because of the small area of the fabric around the pipe, it can quickly clog with fines, effectively blocking groundwater flow into the pipe.

2.6.2 Springhead Drains

Springhead drains are installed to intercept point-source springs, seeps, and shallow water-bearing zones in slopes or on existing landslides. They reduce the possibility for surficial erosion that can reduce stability by undercutting and oversteepening a slope. In addition, they reduce the amount of groundwater that can seep into the surficial colluvial and fill soils, which are often particularly susceptible to landsliding when saturated.

Springhead drains are placed at the point where springs and seeps emanate from the slope, to direct water through pipes to the base of the slope. Springhead drains have filter soils placed at the beginning of the drainpipe to reduce the potential of piping (migration) of soils into the springhead system. Figure 2-9 in appendix shows an example of a typical springhead drain installation.

The installation generally begins with an excavation to expose the seepage zone. The size of the excavation depends on the lateral extent of the seep or spring and on the practical size of a springhead drain. Difficult access on steep, wet slopes may require making excavations using hand tools. The excavation should extend at least 1 foot deeper than the seepage level to form a collection pool. A perforated or slotted 4-inch (minimum) diameter pipe is placed in the excavation perpendicular to the direction of the slope with the pipe ends capped. The pipe is connected to a tight line pipe and a dam of sandbags, concrete, or clay is placed around the connection to seal the leaks and force water into the tight line pipe. The installation must be completed in such a way that the entire seepage zone is backfilled with a free-draining aggregate that is sufficiently permeable to accommodate the anticipated seepage. Often the perforated or slotted pipe is backfilled with pea gravel or other more permeable clean granular aggregate to accommodate the increased flow rates as the collected seepage is concentrated near the collector pipe. Drainage sand and gravel, such as Seattle Type 26 Aggregate, may be suitable for the remainder of the backfill in the seepage zone. The selection of pipe diameter, perforation or slot size, and backfill materials depends on the amount of seepage and the grain size of the surrounding soil. As described in Section 7.2.1, the backfill material(s) must be adequate filters for the surrounding soil to prevent piping.

2.6.3 Drainage Blankets

When fills are constructed on a slope, a drainage blanket should be placed between the fill and the prepared subgrade surface to intercept seepage from the underlying soil and to improve drainage of water that infiltrates from the surface. Fills where a drainage blanket should be considered include toe buttresses, embankment fills, and slope fills placed to restore grades. A drainage blanket consists of a permeable layer of soil that is placed over the prepared subgrade before a fill is placed. Because it is designed to transmit groundwater, a drainage blanket should be designed as a filter for the subgrade and fill soils. Otherwise, piping of fines could plug the filter blanket and/or cause loss of ground.

The drainage blanket should be designed so it is capable of conveying the maximum anticipated seepage and infiltration water without saturating its full thickness. Figure 2-10 in appendix shows an example of a drainage blanket placed beneath an earth buttress fill. The design elements that need to be evaluated for each site include:

- The anticipated groundwater seepage and surface water infiltration rates.
- Permeability of the drainage blanket material and its thickness.
- The maximum distance to an interceptor trench subdrain or outlet.
- Seals to prevent direct surface water infiltration.
- If build on steep slopes, the drainage blanket should be built in benches or steps that penetrate into the natural slope. The drainage blanket should be continuous across the benches and should be graded to drain continuously.

2.6.4 Drilled Drains

Drilled drains consist of generally small-diameter drainpipes installed in drilled holes to a water-bearing soil layer. They are used to lower the groundwater level in a landslide or marginally stable slope where the depth to groundwater is too deep for dewatering using trench subdrains. The main advantage of drilled drains is that they can be installed at virtually any depth. Limitations include relatively high cost and the ability to intercept a sufficient amount of the permeable water-bearing zones to effectively lower the groundwater level. A thorough understanding of the subsurface soil and groundwater conditions is essential in planning a dewatering system using drilled drains. A geotechnical engineer and a hydro geologist should explore the subsurface conditions, evaluate groundwater flow, and perform slope stability studies to develop an optimum drain configuration. The hydro geologist should design the most appropriate drain spacing, well diameter, and well screen size. Pumping tests or other aquifer tests are commonly required to evaluate the effectiveness of proposed drilled drains. If drilled drains are selected as an element in improving the stability of a slope, groundwater monitoring wells should also be installed and monitored before and after drain construction to verify that the drains are achieving the degree of lowering in the groundwater levels desired. These groundwater monitoring wells can also be used to monitor the effectiveness of the system over time.

The three general categories of drilled drains include nearly horizontal drains (commonly called horizontal drains), directionally drilled drains, and vertical drains or wells. Horizontal and directionally drilled drains capture groundwater and drain it away from a sensitive slope area with gravity flow. Vertically drilled drains typically require pumping to remove groundwater, although gravity drainage is possible in certain circumstances, as subsequently described. Figure 2-11 in appendix shows a schematic of the various types of drains. If drilled drains are suitable, the site access limitations, the subsurface conditions, and construction costs typically dictate which system is feasible for a particular site.

Horizontal Drains

Horizontal drains are installed by drilling a nearly horizontal boring from a point at the bottom of a slope. Therefore, access to the bottom of the slope for a large, track-mounted vehicle must be possible for this option. Typically, two or more horizontal drains are radially drilled from one or more points to intercept the water-bearing stratum. The drilled holes extend as far into the hillside as necessary to intercept and lower the groundwater level. The length of drilled drains can be 200 feet or more. They are drilled straight at a constant upward inclination of 2 to 10 degrees from the horizontal, depending on the site access and elevation of the water-bearing zone. The installation technique generally consists of drilling a sub horizontal boring and concurrently placing a steel casing into the hillside. A slotted or screened plastic pipe is then placed inside the casing, which is then withdrawn leaving the plastic pipe in-place. A tight line pipe is attached to the end of the plastic pipe and a low permeability plug installed to force the water into the tight line for conveying the discharge water to a suitable location. Each pipe is generally fitted with individual valves for shutting-off and cleaning-out.

Directional Drains

Directional drains are similar to horizontal drains except that they are typically drilled from the top of the slope using a remotely guided drill to intercept a water-bearing soil layer at a predetermined location. Once the drilled hole reaches the target water-bearing layer, the drill bit continues until it exits the slope at the desired collection point. From there, the water is conveyed in a tight line to a suitable discharge location. The advantage to directionally drilled drains is that access to the bottom of the slope for heavy equipment is not needed. For many landslides or marginally stable slopes, access is not otherwise practical.

The drill rig is typically set up some distance away from the top of the landslide, with an initial drilling inclination on the order of 20 degrees from the horizontal. The position of the drill bit is monitored using an electronic tracking device. The drilling assembly can be steered using a specially tooled drill bit to direct it to the desired dewatering zone and exit point. The allowable radius of curvature of the drill steel limits the amount of steering. Once the hole is completed, it can be reamed if a larger diameter is needed. A slotted or screened plastic pipe, usually 2- or 4-inch-diameter polyvinyl chloride (PVC), is pulled through the drill hole from bottom to top to complete the drain. The discharge is captured at the lower end in a tight line pipe system and conveyed to a suitable discharge location. The upper end of the pipe is capped and encased in a monument at the surface to allow access for maintenance and cleaning.

Vertical Drains

Vertical drains consist of vertically drilled bore holes that extend into or through a water-bearing soil layer and remove the water either by constant pumping or in certain circumstances by gravity flow. Pumped vertical drains are essentially water wells and, as such, are designed and built in the same manner as water wells. Typically, the boring for a well is drilled through the permeable unit where dewatering is planned and into the underlying low permeability soil layer. The well consists of a screened section of well casing that extends through the permeable saturated soil and solid casing extending to the surface. A sand pack is placed between the screen and the native soil to increase the effective diameter of the well and to form a filter between the surrounding soil and the screened well casing. The filter prevents the well from piping fines from the surrounding soil that could cause loss of ground and impair the capacity of the well. Water is removed from the well using a submersible pump that is controlled with a switch activated by rising water level or hydrostatic pressure. Several different pumping system configurations are available. Dewatering wells can intercept water-bearing units that otherwise are not accessible by other types of subsurface drainage. However, they require continual pumping and maintenance, which can be costly. In addition, a reliable power source is essential because of the likelihood of power outages during wet stormy periods. Backup power systems require frequent maintenance and testing to ensure that they will function when the normal power system is interrupted.

A vertical gravity drain is installed in a similar manner as the pumping well, but instead of using a pump to remove water, the well drains groundwater to an underlying layer of permeable soil. This type of system requires specific subsurface conditions to be practical. These are:

1. The water-bearing layer that is reducing the slope stability must overlie a lower permeability layer (aquitard).
2. The aquitard must be underlain by a zone of permeable soil (e.g., sand or gravel).
3. The lower permeable zone must be below the level of slope instability.
4. The lower permeable zone must be able to drain the upper water-bearing layer, i.e., it must have sufficient permeability, thickness, and there must be a sufficiently large hydraulic gradient.

The design of this type of system requires detailed knowledge of the hydraulic characteristics of the entire system. A hydrogeologist is typically required to evaluate the soil parameters and design the well system. Another concern associated with vertical gravity drainage is the potential for cross contamination between upper and lower aquifers. If the groundwater in the upper soil layer is contaminated, vertical drainage into an underlying aquifer would be prohibited by environmental regulations. Even if the upper aquifer is not contaminated, the Washington State Department of Ecology or other local environmental regulatory agencies may require work to demonstrate that the underlying aquifer would not be degraded.

2.6.5 Other Subsurface Drainage Systems

Numerous other subsurface drainage systems have been used to lower groundwater levels in landslides and in marginally stable slopes. These other systems typically are appropriate for specific subsurface geologic and groundwater conditions and are not widely applicable. Some systems have largely been replaced because of technological advances. For example, during the Depression, several U.S. Works Progress Administration (WPA) projects were undertaken to install drainage in landslide areas. In many cases, the drainage consisted of hand-excavated tunnels that were subsequently backfilled with drain pipe and sand and gravel. Today, many of these drains would be installed by horizontal or directional drilling. Still drainage tunnels have specific, if limited, use for subsurface drainage. Other subsurface drainage systems include: electro-osmosis, vacuum dewatering, and siphoning. Because of the limited and site specific applications, these methods are not discussed in this report.

2.6.6 Monitoring and Maintaining Subsurface Drainage Systems

Subsurface drainage systems are only effective if they lower the groundwater level at least to the level assumed for design and if they maintain the lowered groundwater level. Therefore, we recommend performing regular maintenance and installing a monitoring system so that the effectiveness of a subsurface drainage system can be monitored. The type of monitoring depends on the site conditions, the type or types of subsurface drainage system(s) used, and the degree of reliability required. Maintenance includes clearing vegetation from outlet pipes and tightlines, inspecting and repairing damage to surface installations, removing accumulated sediment from catch basins, and jetting pipes to remove sediment and encrustation.

Subsurface drainage systems are usually monitored by measuring the groundwater level in one or more monitoring wells and measuring the discharge rates from drain outlets.

The continuity of a drain line can also be evaluated by adding water at an uphill cleanout location and observing the flow at a downhill discharge location. However, this type of test should only be performed during the dry summer season. The groundwater level in monitoring wells can show that the drainage system is lowering the groundwater to the levels assumed in design. Monitoring wells should be installed before the subsurface drainage system is installed to establish pre-construction groundwater level(s). Often the monitoring wells that were installed during the initial site explorations can be used for long-term monitoring. After the subsurface groundwater drainage system is installed, the groundwater levels should be monitored on a regular basis to evaluate the performance of the drainage system, including its response to seasonal rainfall events. The measurement and data recording interval should be determined for each site. Depending on the complexity and criticality of the subsurface drainage system, an automated data recording system may be justified. Once the groundwater response to seasonal and rainfall events is established, groundwater level monitoring should be conducted at least once on an annual basis thereafter. Unanticipated changes in groundwater levels typically show the need for cleaning or other maintenance. The discharge rates from subsurface drains should also be measured and recorded. Declines in the discharge rate may indicate buildups of encrustation or sediment that reduce the effectiveness of the system.

Subsurface drainage systems require regular maintenance to perform as designed. Maintenance should start by designing surface installations that are protected from damage. For wells, this could be accomplished by installing guard posts and steel monuments to prevent vandalism and accidental damage. All surface installations, such as wells, drains, and tight lines, should be placed in locations where they can be easily found. Vegetation around these installations should be regularly trimmed to allow inspection for deterioration, breaks, leaks, and other damage. If groundwater monitoring and/or discharge rates indicate a decline in the performance of the subsurface drainage system, it should be cleaned by flushing, jetting, or other appropriate means.

CHAPTER 3

METHODOLOGY

There are 15 experiments were conducted for this project. Methodologies of this project consist of four major parts that involve the soil samples from Bukit Kledang. These are:

- Soil sampling and field data observation
- Soil properties and engineering properties
- Simulation using laboratory modeling
- Analysis using Slope/W software

3.1 SOIL SAMPLING AND FIELD OBSERVATION

The samples of the soil are from weathering of granite. The soil samples were collected at the toe of Bukit Kledang. The top layer of the soil was removed and the soil was taken from the A and B horizons. The soil samples were taken using spade, hoe and trowel and stored into a gunny sack of 40kg each every time the soil sample was taken. There are all together 5 collection periods. The total soil sample was about 200kg. During the soil sampling, bulk density test was conducted by augering the soil to about 1m.Then a steel cylinder was pushed into the soil until it's full. This is to make sure the soil sample taken for this experiment remains undisturbed. After filling the cylinder with soil, the cylinder was covered to keep the moisture content of the soil. The soil samples were then taken to the geotechnical lab for air drying process and for soil analysis on engineering properties. Field observations were done by climb up the hill with a car and taking picture where necessary such as seepage visibility, water fall at the hill and slope failure occurrence. These pictures show the condition of the hill with places of high groundwater level. The field observation was done by climbing until the seismographic station because the road towards the top of the hill is so narrow to climb and dangerous.

Figure3.1. Augering the soil sample

Figure3.2. Collecting soil for bulk density test

3.2 LABORATORY EXPERIMENT

Laboratory experiments were conducted to analyze the soil and experiments involved are as follows:

- Determination of moisture content, specific gravity , type of soil using particle distribution test and hydrometer test and bulk density test for physical properties of the soil
- Atterberg Limit (Plastic limit and liquid limit analysis)
- Permeability test of the soil
- Direct Shear Box Test for determine the cohesion, friction angle and peak shear stress of the soil

3.2.1 Physical Soil Properties

Physical soil properties was analyzed five parameters were determined:

- Specific gravity by small pyknometer method
- Particle distribution by dry sieving test and hydrometer test
- Bulk density by steel cylinder auger
- Moisture content by oven-drying method

Specific gravity was determined by the small pyknometer method. Small pyknometer method was used for soils consisting of clay silt and sand-sized particles. The procedure of the test was based on laboratory manual according to the standard of BS1337:Part2:1990:8.2. This experiment took 24 hours to get the results as the small pyknometer need to be kept standing at least for 24 hours at room temperature.

Particle distribution of the soil was determined using dry sieving test and hydrometer test. The dry sieving test was to determine the gravel to sand size distribution. Dry sieving test is suitable for soils containing insignificant quantities of silt and sand. This test follows the standard of BS1337:Part2:1990. As for the hydrometer, the purpose is to determine the size distribution of fine particle that is smaller than 63μm and following the standard of BS1337:Part2:1990:9.6. Hydrometer test was conducted to determine the particle distribution by taking the reading in a period of 24 hours. The elapsed time between one to another was taken in a set as below.

Table3.1. Time data recorded for hydrometer test.

Number of reading	Time data recorded
1	30 second
2	1 minute
3	2 minute
4	4 minute
5	8 minute
6	30 minute
7	2 hours
8	4 hours
9	8 hours
10	24 hours

Before the data recorded, calibration of the hydrometer was conducted for the correction of the data. After the data was recorded, calculation was done for the percentage of the soil distribution.

Figure3.3. Sieving pan for dry sieving test

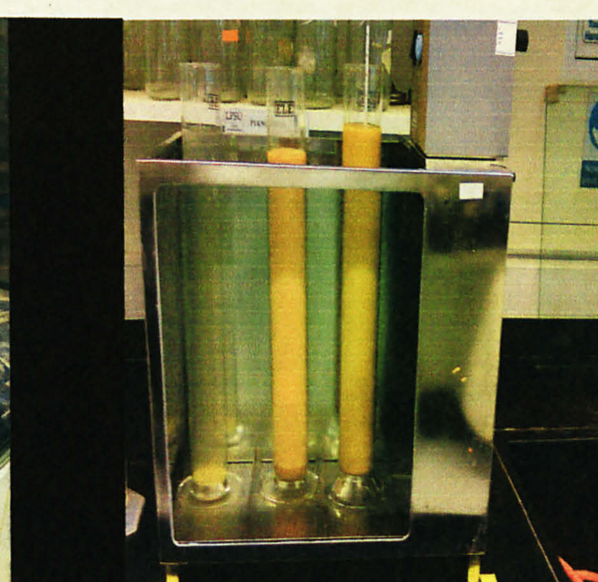


Figure3.4. Hydrometer Test in progress

Bulk density of the soil was determined using the collected soil sample. This was done by calculating the volume and weight of the soil. The soil was extruded by a machine extruder from the steel cylinder auger. The volume of the soil was record by measuring the length of the soil and the diameter of the soil. Next, the soil was weighed to record the weight of the soil sample. Then the soil was analyzed for their moisture content by oven drying method. The soil was kept in the oven at 100°C for 24 hours to eliminate the moisture of the soil.

3.2.2 Atterberg Limit

The experiments of atterberg limit are plastic limit and liquid limit (cone penetrometer method). Plastic limit is the empirically establish moisture content at which a soil become too dry to be plastic. It is used together with the liquid limit to determine the plasticity index which when plotted against the liquid limit on the plasticity chart provides a means of classifying cohesive soils. This both experiment follow the standard of BS1337:Part2:1990:4.3/4.4

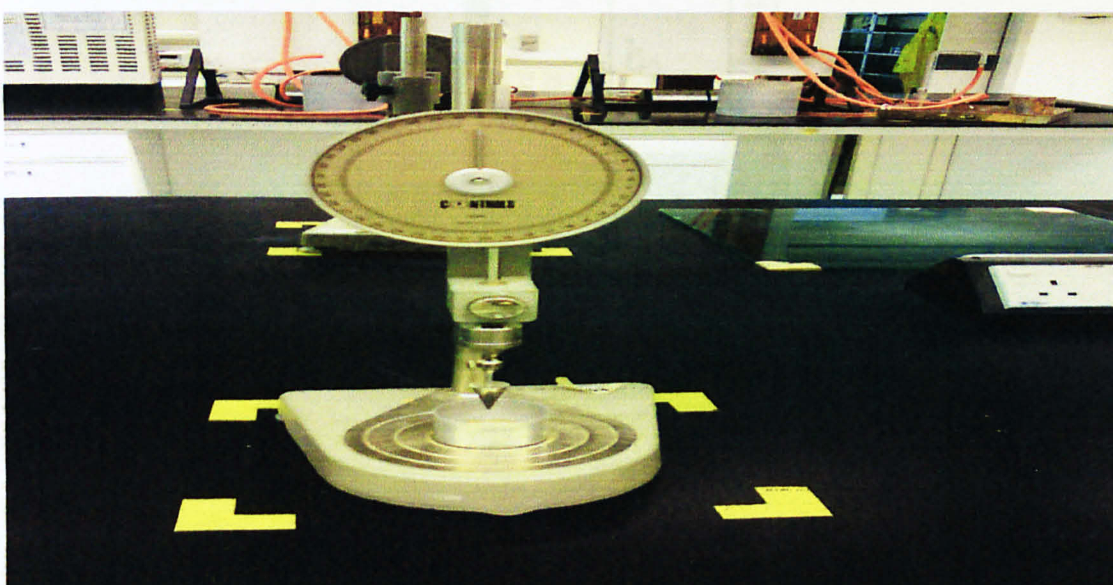


Figure3.5. Liquid limit apparatus

3.2.3 Permeability of Soil

Permeability if soil is a measure of its capacity to allow flow of water through the pore spaces between solid particles. The degree of permeability is determined by applying a hydraulic pressure gradient in sample of saturated soil and measuring the consequent rate of flow. The coefficient of permeability is expressed in velocity.

The test was done using falling head permeability test. This is suitable for soil that have small particle size because the soil having smaller value of permeability. The cross section of falling head permeability test is shown in figure7.

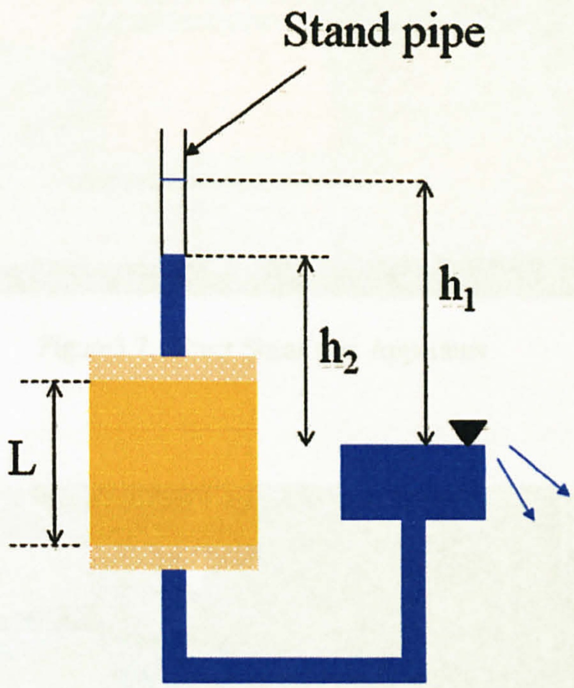


Figure3.6. Cross section for falling head permeability test

3.2.4 Direct Shear Box Test

In direct shear box, a square prism of soil is laterally restrained and sheared along a mechanically induced horizontal plane while subjected to pressure applied normal to that plane. The shear resistance offered by the soil as one portion is made to slide on the other measured at regular intervals of displacement. Failure occurs when the shearing resistance the maximum value which the soil can sustain. By carrying out the test on a 3 similar specimens of the same soil under different normal pressure, the relationship between measured shear stress at failure and normal applied is obtain.

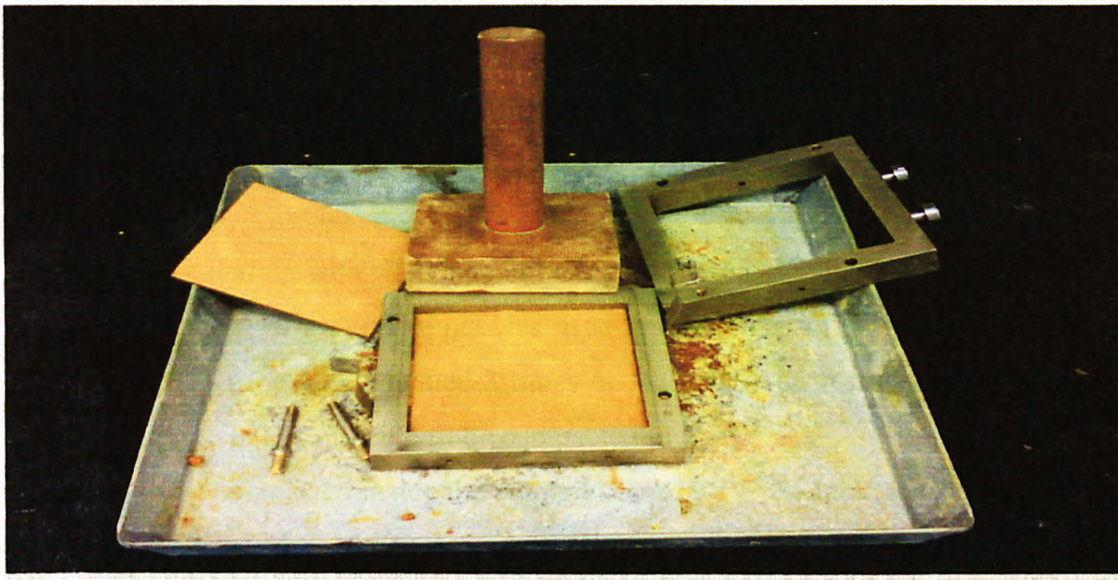


Figure3.7. Direct Shear Box Apparatus

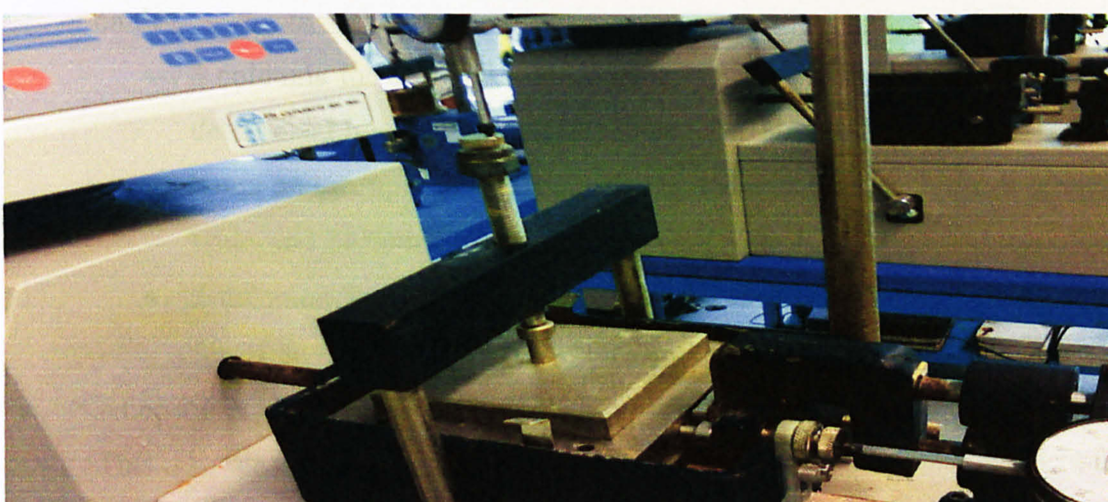


Figure3.8. Direct shear box test in progress

3.3 LABORATORY MODELING

Physical laboratory models were conducted using soil column and sprinkler in order to determine the moisture content of the soil due to the effect of rain and rising groundwater level. The flows of the laboratory modeling are as below:

i. The soil sample was air dried and separated from foreign element (other than soil)

ii. Models were designed with PVC columns of 1meter height. The column was designed and constructed as in figure 3.9 and figure 3.10.

iii. The construction of column was done by sawing the top part of the column to make a gradient at the ratio of 1:1.5. This was to simulate the cut slope of the origin soil. The diameter of the column is 15cm. The bottom part was covered with net tied up with wire. The bottom part of the column was provided with four holes. Next, two layers of net was used to cover the bottom part and tie up with wire. Then the wire was tied 3 times of the column to make sure the net is firm enough to hold soil that will be compacted in it.

iv. Soil was compacted inside the column according to the bulk density of the soil by compacting with 25 blows in 5 layers. The compaction was done using a 4kg weight borrowed from structural laboratory. The net provided at the bottom part act as drainage of the soil while keeping the soil inside the column.

v. There are 5 column prepared for the modeling experiment that was operated in 2 modes.

- 3 columns for artificial rainfall using sprinkler. The rainfall amount was determined by using a metal cylinder. The sprinkler was operated for 24 hours and sample of water inside the cylinder was measure 6 hours, 12 hours and 24 hours.
- 2 columns for artificial groundwater level. The columns were immersed for 24 hours in two pails of water with the level of 10cm and 20 cm.

vi. After the experiment, the columns were cut into 5 sections each of 20cm height. This is to see the cross section of the soil and check the soil moisture content from each section of the column.

vii. The shear strength of the soil was determined using direct shear box method. The cohesion and friction angle were determined from the results of the direct shear box experiment. This analysis using Slope/W software. The direct shear box test determines the cohesion and friction angle by taking the greatest reading of force dial gauge before the soil fail with the normal stress of 100kN, 200kN and 300kN. The moisture content will be determined by oven drying method. The soil from each section needs to go through this experiment to see the different of soil properties under different moisture content.

viii. Interpretation of the result and analysis of the data was carried out using Slope/W analysis

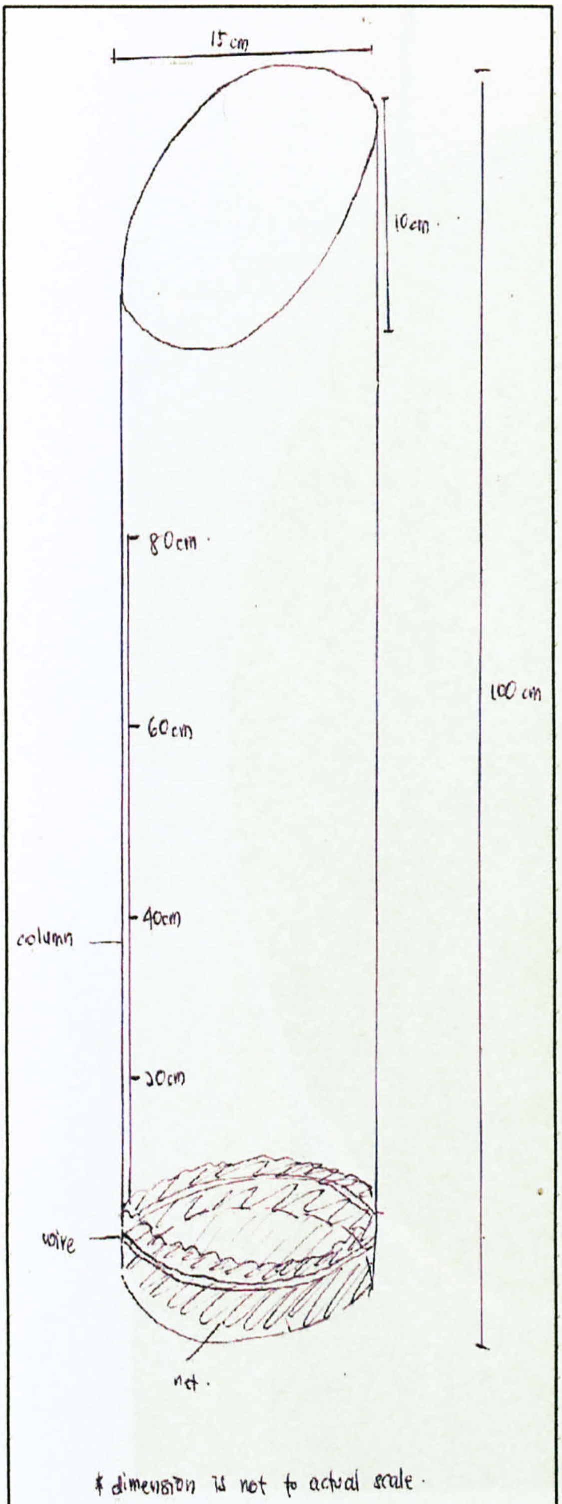


Figure3.9. Sketch design of the laboratory model using PVC column

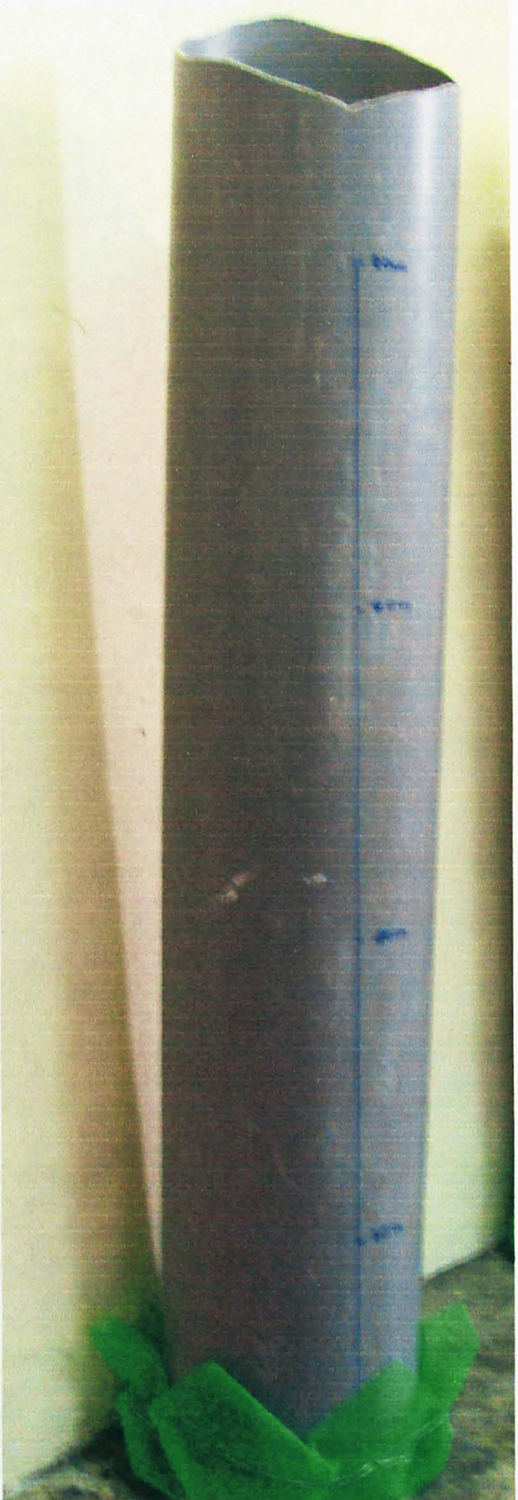


Figure3.10. Model of the PVC column

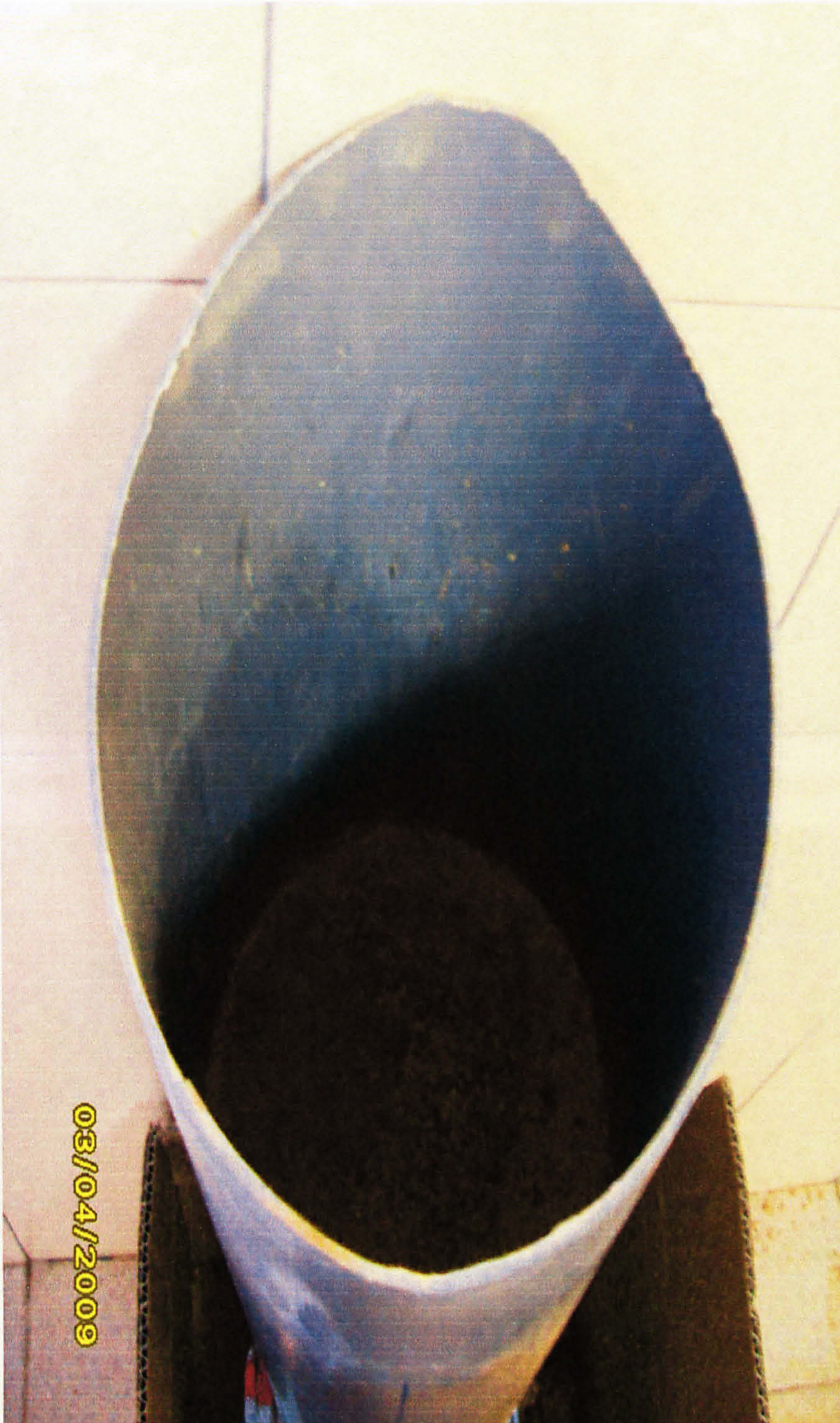


Figure3.11. The compacted soil in the PVC column

Figure3.12. Laboratory model for artificial rainfall using sprinkler in progress

Figure3.13. Simulation of collecting rainfall

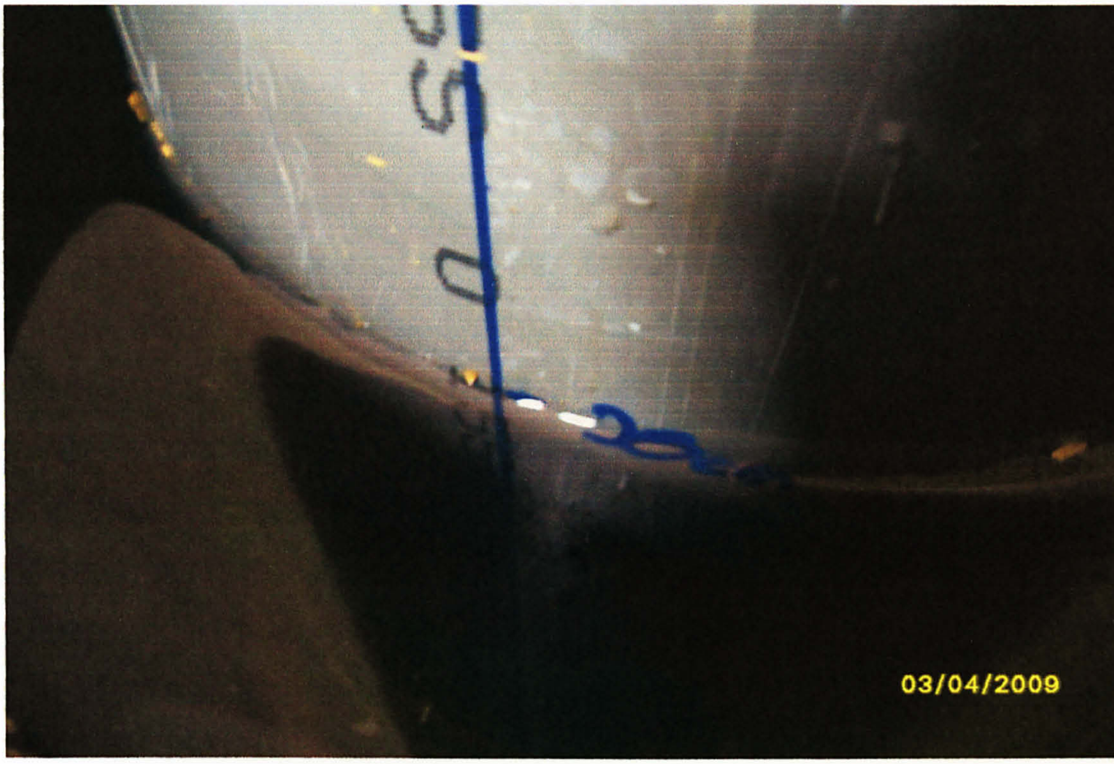


Figure3.14. Laboratory model of artificial groundwater at 20cm in progress

3.4 STABILITY ANALYSIS USING SLOPE/W

Slope stability analysis was done to determine the stability of a slope while load was given. On this analysis using SLOPE/W software, data requirements that need to be fulfill is the soil profile which is:

- Soil types
- Soil Properties
- Properties of slope (ex: slope height, width)
- Shear Strength (ex: cohesion, friction angle)
- Groundwater Table
- Unit weight

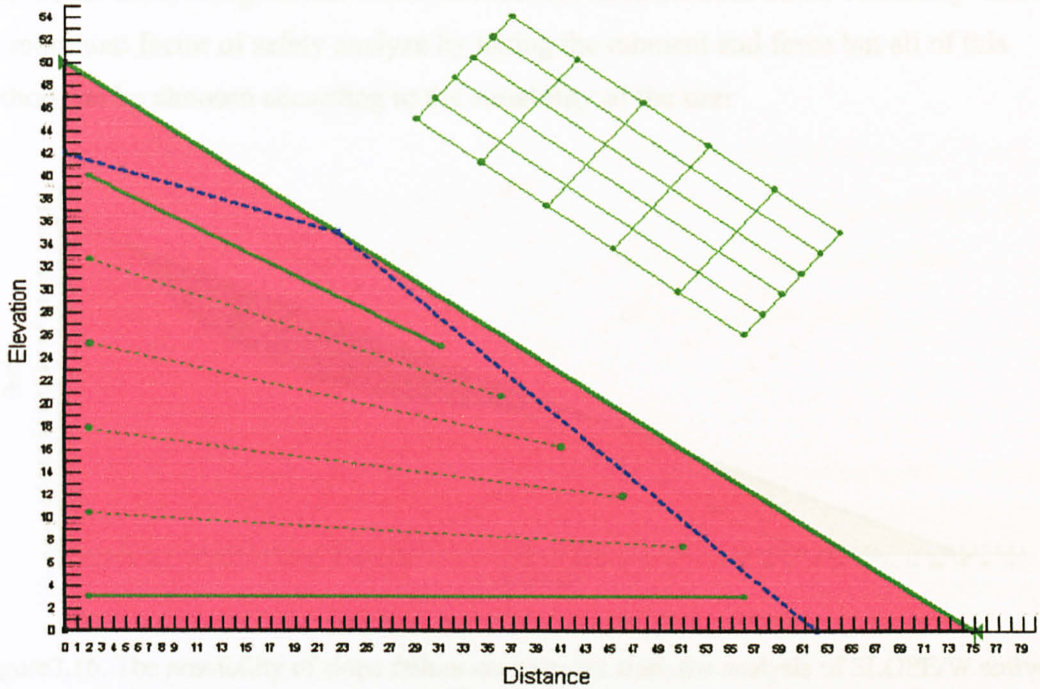


Figure3.15. Example of analysis using SLOPE/W

The main thing that needs to be determined using SLOPE/W is the slope factor of safety (FOS) where on this software, many methods can be used to determine FOS which include:

- Ordinary or Fellenius
- Bishop's Simplified
- Janbu's Simplified
- Spencer
- Morgenstern-Price
- Corps of Engineers
- Lowe-Karafiath
- GLE (General Limit Equilibrium)
- Finite-Element Stress

Most of the time, Morgenstren- Price method was used because of the efficiency due to the minimum factor of safety analyze by taking the moment and force but all of this method can be choosen according to the suitability of the user

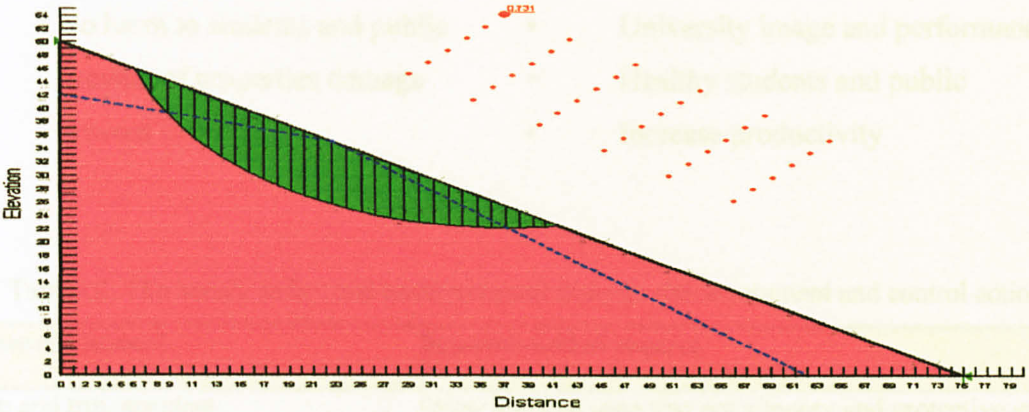


Figure3.16. The possibility of slope failure occurrences from the analysis of SLOPE/W software (refer to the Figure 3.15 data)

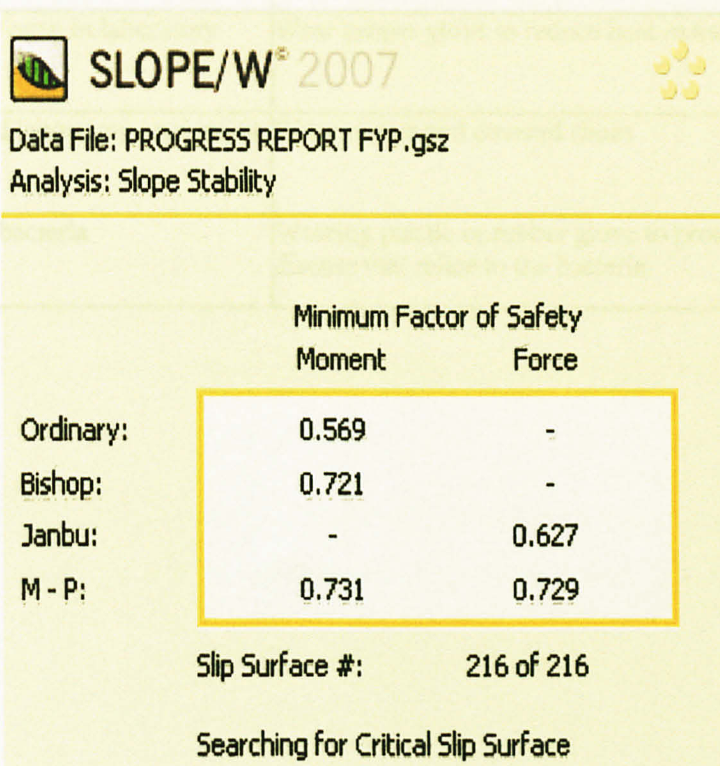


Figure3.17. Analysis of the minimum factor of safety (refer to Figure3.15)

3.5 HEALTH, SAFETY AND ENVIRONMENTAL

The objective of health, safety and environment are:

- Accident prevention
- No harm to students and public
- Prevent of properties damage
- Prevent loss event
- Environment friendly protection
- University image and performance
- Healthy students and public
- Increase productivity

Table3.2. The health, safety and environmental analysis risk assessment and control action

Potential hazard	Recommended control
Slip and trip, accident	Wear suitable shoe that not slippery and protective gear
Extreme heat and cold	Wear proper attire and equipment (raincoat and hat)
Extreme heat by oven in laboratory	Wear proper glove to reduce heat radiation
Falling hazard of laboratory equipment	Wear proper and covered shoes
Microorganism bacteria	Wearing plastic or rubber glove to protect hand from any disease that relate to the bacteria

CHAPTER 4

RESULT AND DISCUSSION

4.1 ASSESMENT OF LOCAL AND DESIGN REPORT

Based on the assessment of the previous report, several causes of landslide have been identified. The main cause of landslide or slope failure is by water or slope saturation that reduces the strength of the soil and lead to slope instability. Groundwater table has a big impact on the slope stability. The fluctuation of groundwater an affect on slope stability by changing the effective stress and thus resistant to shear strength by creating seepage and by running as an agent of weathering and erosion to promote dissolution in soluble rocks, swelling in expensive clays and erode of fine particles from weak bond cement deposits.

In raining season, movements of water through soil create seepage force and reduce shear strength. Ground water flow is capable to transfer minerals grains and can be defined as internal erosion. This would lead to overlaying layers collapse and promote a slope failure.

Ground water recharge is essential to interpreting changes in pressure head. The pressure head increasing until the end of rainfall and after rainfall stop, pressure head decrease rapidly.

4.2 SOIL SAMPLING AND FIELD OBSERVATION

The site location of the soil sampling is at toe of Bukit Kledang, Ipoh. This place is near the residential area and has a road and drainage facilities, rest hut for climber to climb the hill. The place has much water seepage through the bedrock of the hill from the top to down of the hill. It also has water stream and small river at the middle height of the hill. At almost the top of the hill, it has a seismographic station for Ipoh. Along the road climbing the hill, a minor slope failure occurs and it was covered with plastic canvas as protection so that the slope failure would not be spreading. This was probably due to the site condition is having a high groundwater table. Seepage and water stream can be found just by the road side. Therefore a control action for landslide should be taken so as to control the occurrences of the landslide. This can be done by reducing the groundwater table so that the seepage occurrence would be reduces and the risk of landslide can be under control.

The soil taken is soft and not very stiff. This was because the soil was wet and the condition at Bukit Kledang always having rain and the groundwater is high due to the visibility of seepage and small waterfall. Therefore all the soil samples needs to be air dried prior to the determination of the soil properties at normal condition.

Figure4.1. Slope failure occurence

Figure4.2. Seepage visibility

4.3 LABORATORY EXPERIMENTAL RESULTS

Several laboratory experiments were conducted on soil properties. Result of the experiments are as follow:

4.3.1 Physical Soil Properties

Specific gravity was determined by using a small pyknometer method. The result for specific gravity of Bukit Kledang soil samples was found to be 1.81.

The particle distribution of the soil was analyzed by dry sieving and hydrometer methods. The dry sieving test results are shown in figure 4.3. The results indicate that the particle distribution of the soil is poorly graded of medium sand. As for the hydrometer test, the results is tabulated below in table 4.1.

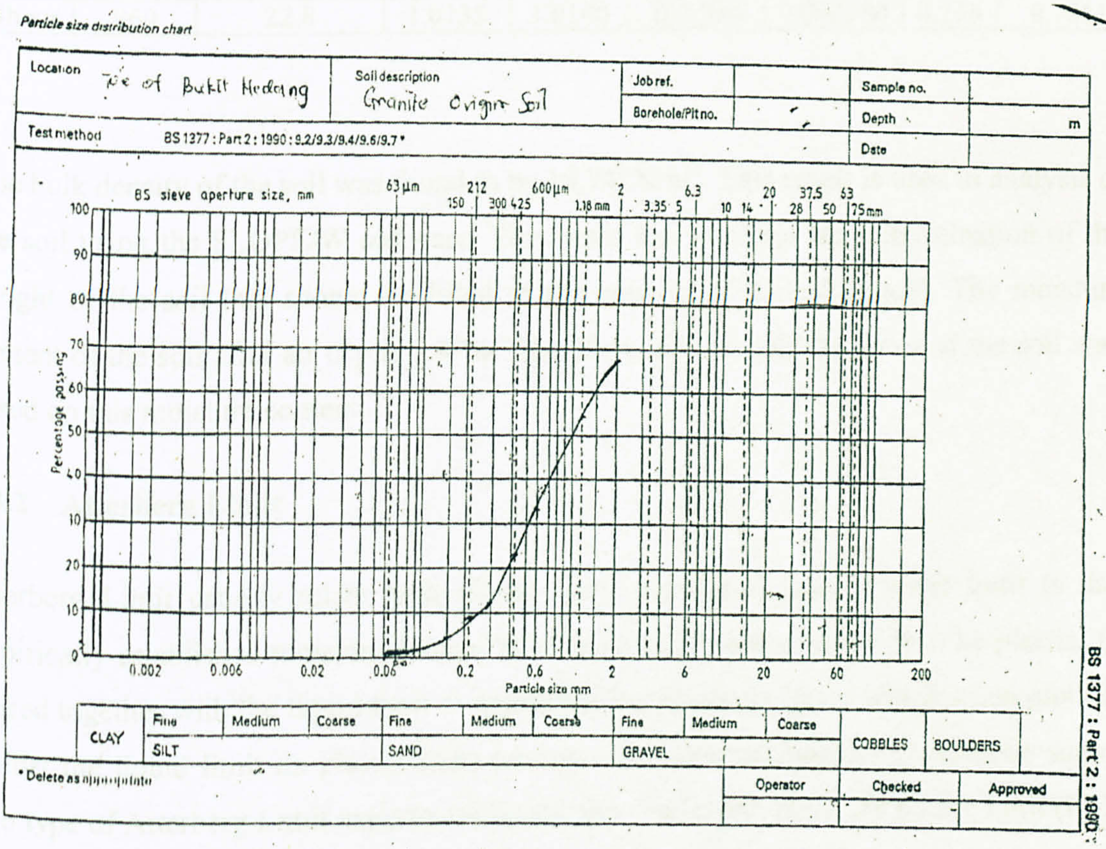


Figure4.3. Particle distribution test by dry sieving method result

Table4.1. Result of hydrometer sedimentation test

Calibration Data	
Meniscus correction	0.005cm
Reading in dipersant (Ro')	0.9977
Calibration equation	$H_r = H + ½ [h- V_hL/90]$
Dry mass of soil (m)	50g
Particle density (ρ_s)	1.81 Mg/ m^3
Viscosity of water at 22.9 °C (η)	0.9354 mPa.s

Test Data								
Time	Elapsed time, t (min)	Temperature, T (°C)	Reading (R_h')	$R_h' + C_m = R_h$	Effective depth, H_r (mm)	Particle Density, D (mm)	$R_h' - R_0' = R_d$	Percentage finer than D, K (%)
30sec	0.5	22.9	1.0265	1.0270	116.364	0.087396	0.288	1.2871
1min	0.5	22.9	1.0260	1.0265	118.316	0.088125	0.283	1.2648
2min	1	22.9	1.0250	1.0255	122.220	0.063334	0.273	1.2201
4min	2	22.9	1.0232	1.0237	133.832	0.046863	0.255	1.1396
8min	4	22.9	1.0215	1.0220	135.764	0.033375	0.238	1.0637
30min	22	22.9	1.0187	1.0192	147.414	0.014829	0.210	0.9385
2hour	90	22.8	1.0156	1.0161	159.180	0.007619	0.179	0.8000
8hour	360	22.9	1.0140	1.0145	165.079	0.003879	0.163	0.7285
24hour	960	22.8	1.0135	1.0140	167.056	0.002390	0.158	0.7061

The bulk density of the soil was found to be 18.74kN/m^3. This result is used in analysis of the soil using the SLOPE/W software. The result was used for the determination of the weight of the soil that should be filled in the column of the lab model. The moisture content of the soil after air dry is 7.87%. Therefore, all the other analysis of the soil was based on this moisture content.

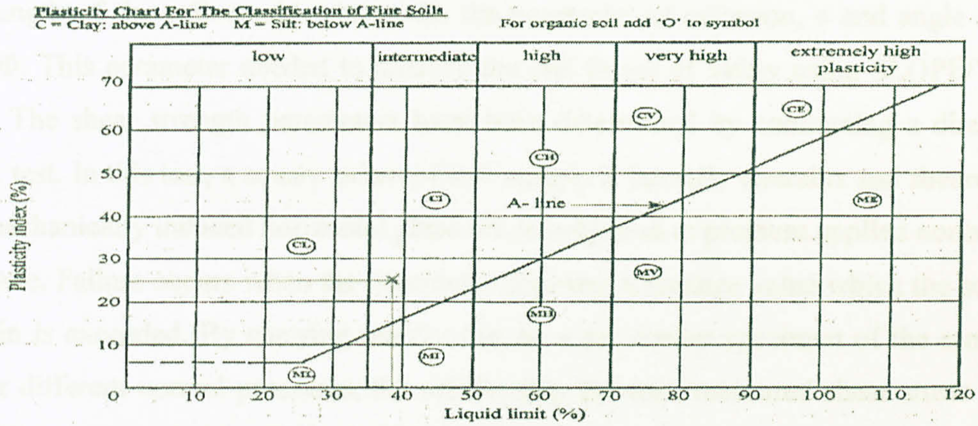
4.3.2 Atterberg Limit

Atterberg Limit usually relate with plastic limit and liquid limit. Plastic limit is the empirically established moisture content at which a soil becomes too dry to be plastic. It is used together with the liquid limit to determine the plasticity index which when plotted against the liquid limit on plastic chart provides a means of classifying cohesive soils. Two type of Atterberg Limit experiment have been conducted, there are plastic limit (PL) and liquid limit (LL).

As for the plastic limit, the result was found to be 25% and the liquid limit was 39%. The moisture content of the soil is 7.87%. Therefore plasticity index is 14% as calculated using the equation 1 below.

Plasticity Index (I_p) = LL – PL - Equation 1

From the plasticity chart for the classification of fine soils in figure 13, the classification for the soil sample can be determine. This soil sample is in the clay group with intermediate plasticity.



Sub-group symbols in the British Soil Classification System

	Primary letter	Secondary letter
Coarse-grained soils	G = GRAVEL S = SAND	W = well graded P = poorly graded Pu = uniform Pg = gap graded
Fine-gained soils	F = FINES (undifferentiated) M = SILT C = CLAY	L = low plasticity I = intermediate plasticity H = high plasticity V = very high plasticity E = extremely plasticity
Organic soils	Pt = PEAT	O = Organic

Figure4.4. Plasticity chart for the classification of fine soils

4.3.2 Permeability

The permeability experiment by falling head test indicates the result as 0.0011cm/s.

$$k = 2.303 \frac{aL}{At} \log_{10} \frac{h_1}{h_2}$$ - Equation 2

The permeability test conducted is to have the constant value of the hydraulic conductivity of the soil sample. This is because the pumping method induces at the slope should be calculated in order to find the distance of the vertical drilled drain from one to another. The limitation of relatively high cost need to be considered as the distance and calculation should be accurate, so that the cost is not wasted due to some technical error.

4.3.3 Direct Shear Box

Shear strength of the soil would relate with the parameter of cohesion, c and angle of fraction, Φ. This parameter needed to analyze the soil factor of safety using SLOPE/W software. The shear strength parameters have been determined by conducting a direct shear box test. In this test, a square prism of soil sample is laterally restrains and sheared along a mechanically induced horizontal plane while subjected to pressure applied normal to that plane. Failure occurs when the maximum shearing resistance value which the soil can sustain is exceeded. By carrying out the test on a set similar specimen of the same soil under different normal pressures, the relationship between measured shear stress at failure and normal applied is obtained. The parameter of shear strength was determined is shown in figure 4.5. The cohesion friction angle of the soil was found to be 5kPa and 25° respectively.

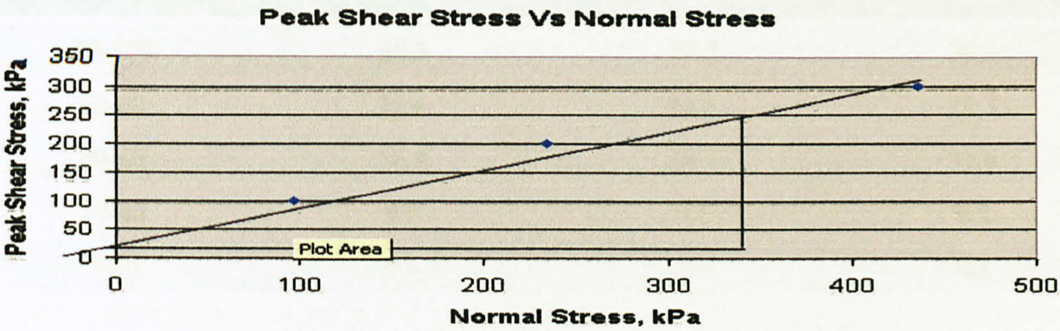


Figure4.5. Graph of peak shear stress versus normal stress for direct shear box test.

Table4.2. Summary of Soil Properties of Bukit Kledang

SOIL TYPE	SILTY CLAY
SPECIFIC GRAVITY	1.81
PERMEABILI TY (cm/s)	0.0011
UNIT WEIGHT (kN/m^3)	18.74
COHESION (kN/m^2)	5
FRICTION ANGLE, ø (degree)	25

4.4 LABORATORY MODELING EXPERIMENT RESULTS

The result of the laboratory modeling is according to the moisture content of the entire column under the two modes of experiment. Results of the artificial rainfall and artificial groundwater level experiments using five columns are as below.

Table4.3. Moisture content of the laboratory model for artificial of rainfall

HEIGHT OF COLUMN (cm)	6 HOURS OF INFILLTRATION OF WATER (%)	12 HOURS OF INFILLTRATION OF WATER (%)	24 HOURS OF INFILLTRATION OF WATER (%)
80-100	28.8	29.3	30.0
60-80	28.6	29.1	29.7
40-60	28.4	28.9	29.6
20-40	9.0	9.5	9.8
0-20	7.9	8.0	8.2

Table4.4. Moisture content of the laboratory model for artificial of groundwater table

HEIGHT OF COLUMN (cm)	10cm OF WATER LEVEL (%)	20cm OF WATER LEVEL(%)
80-100	**6.4**	**6.5**
60-80	**7.4**	**7.4**
40-60	**7.9**	**8.3**
20-40	**10.4**	**26.8**
0-20	**25.6**	**30.2**

From table 4.3, the results show that the top 60cm of the column is having high moisture content due to the infiltration of the sprinkler water through the soil. However towards the bottom of column the moisture content of the soil is almost the same as the air dry moisture content of the sample. This shows that the soil after 24 hours period of artificial rainfall the soils at the bottom having are still low moisture content with low shear stress. Thus the soil and can remain stable.

Table 4.4 shows that the moisture content of the bottom part is higher than the top. This is because the columns were immersed in the container for 24 hours. The pore water pressure increases the moisture content of the soil above the saturation point. This show that soil with high groundwater level, can affect the moisture content of the upper elevation of the soil. This reduces the soil shear strength and increase the shear stress and having lower cohesion due to the water is lowering the friction of resistance.

Therefore, the results of the laboratory modeling can be used to represent the pumping method hypothesis i.e. by induced pumping at slope can reduce the groundwater level and will reduce the occurrence of landslide.

4.5 ANALYSIS SLOPE STABILITY USING SLOPE/W

Slope stability analysis was conducted by using slope/w software to determine the factor of safety of the slope. All the required parameters were identified by the laboratory experimental process. To start the analysis the height and the width of the slope was set. The ratio of the height and width was assumed to be 1:1.5 that is the normal ratio for cut slope. The height of the slope was determined based on the 4 set data as below:

	Height (m)	Width (m)
a)	100	150.0
b)	75	112.5
c)	50	75.0
d)	10	15.0

Using the result of the cohesion, friction angle, unit weight, and assume of high ground water table due to water stream and seepage visibility. The analyses of the slope are as follow:

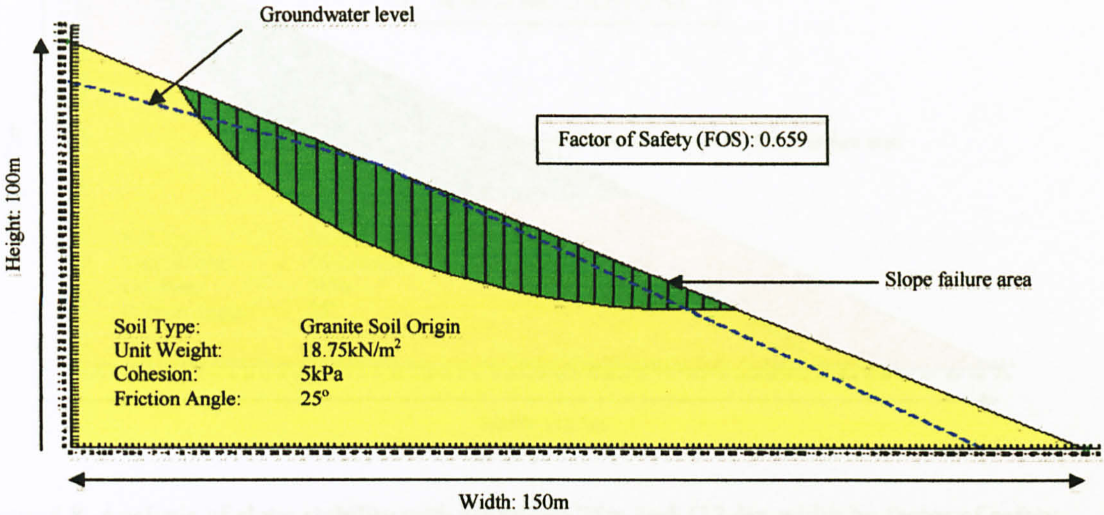


Figure4.6. Analysis of slope stability with set height 100m and 150m width by factor of safety using SLOPE/W software with high groundwater level

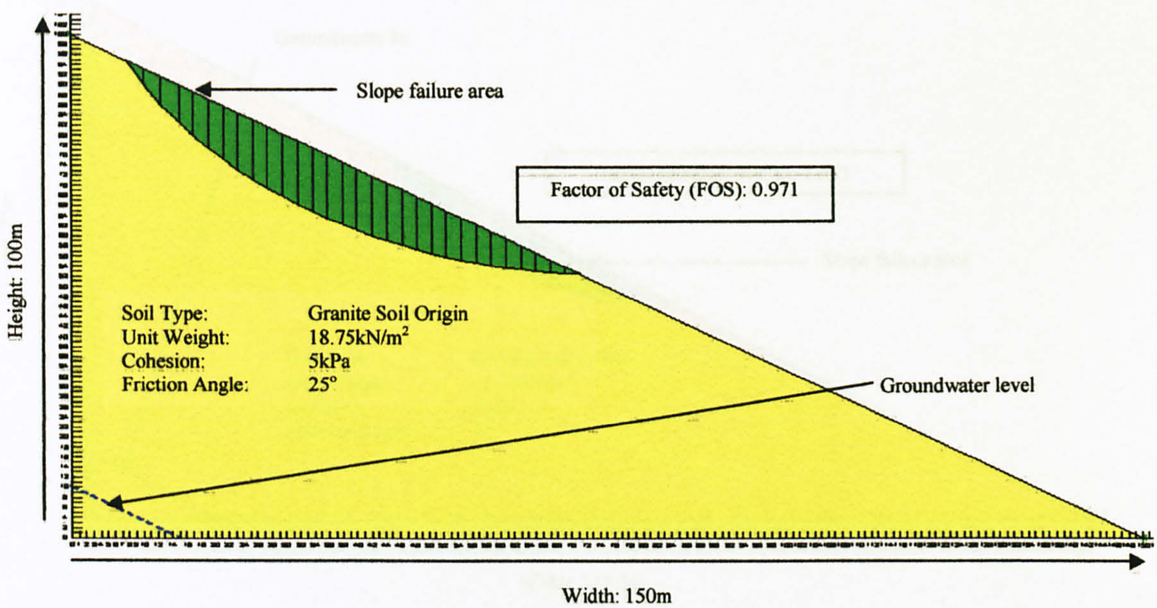


Figure4.7. Analysis of slope stability with set height 100m and 150m width by factor of safety using SLOPE/W software with low groundwater level and induced of pumping method

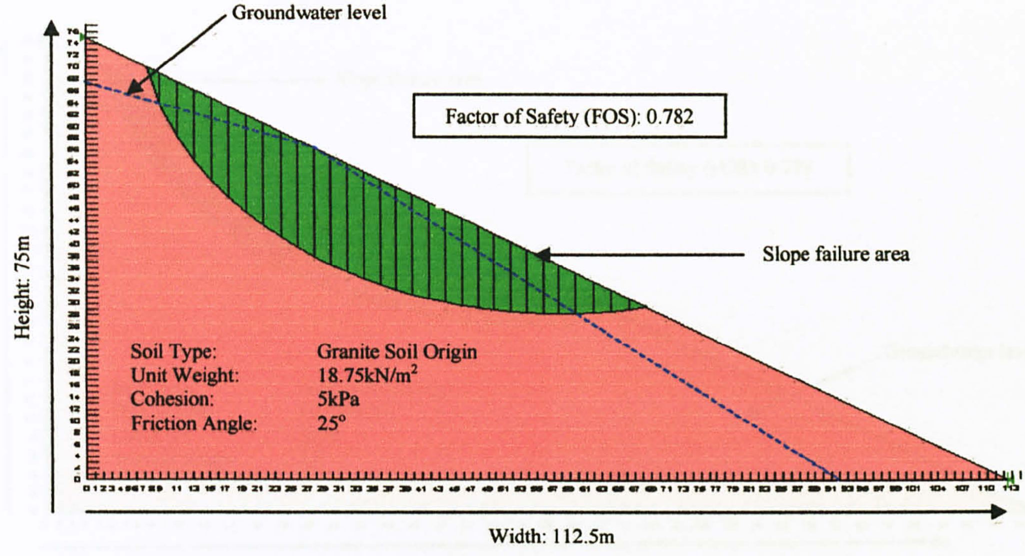


Figure4.8. Analysis of slope stability with set height 75m and 112.5m width by factor of safety using SLOPE/W software with high groundwater level

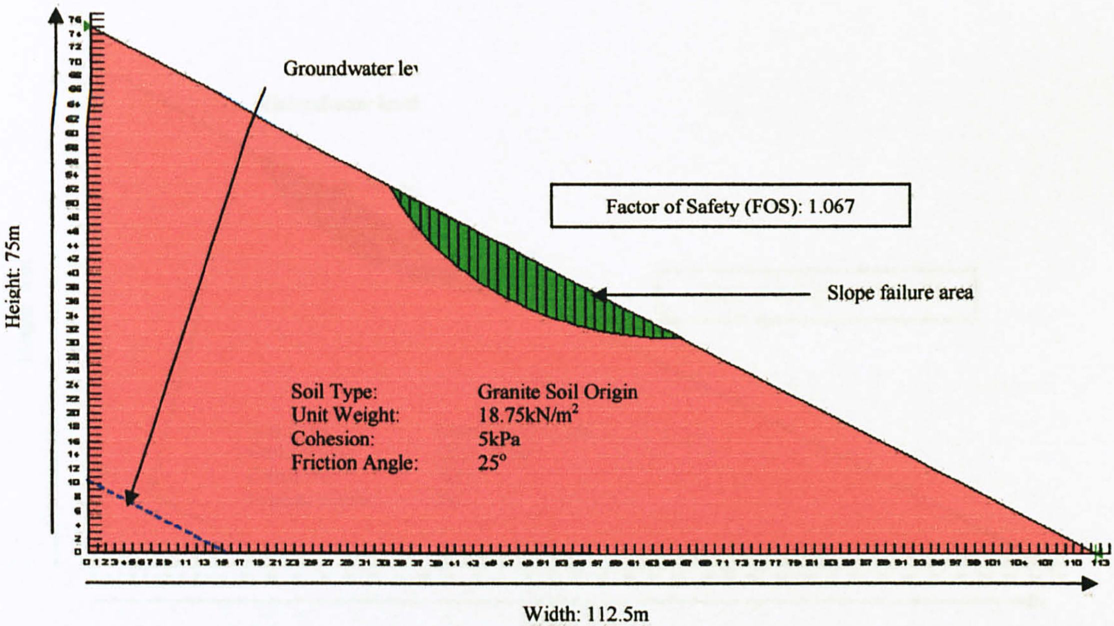


Figure4.9. Analysis of slope stability with set height 75m and 112.5m width by factor of safety using SLOPE/W software with low groundwater level and induced of pumping method

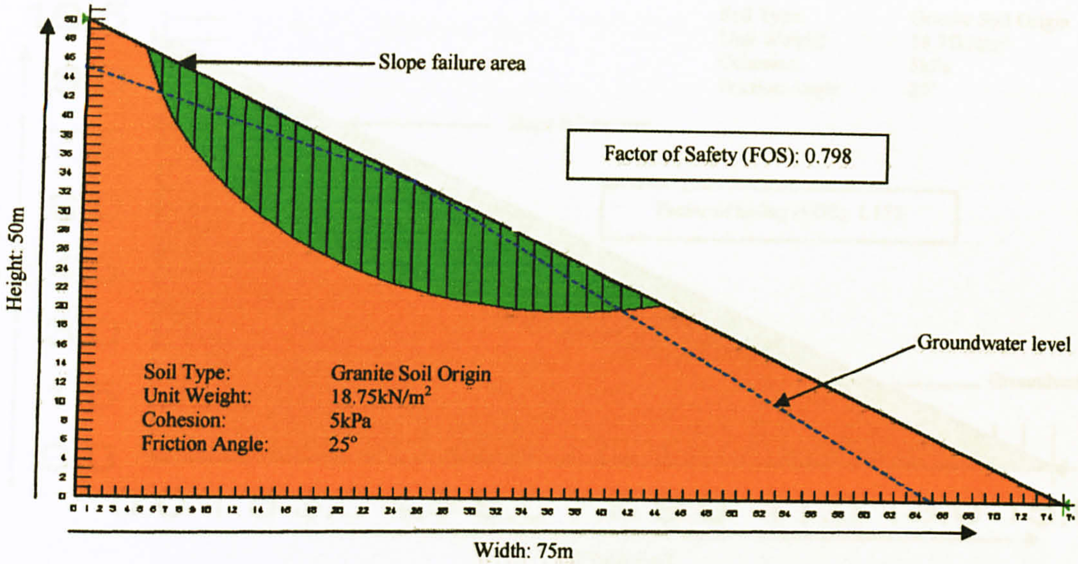


Figure4.10. Analysis of slope stability with height 50m and 75m width by factor of safety using SLOPE/W software with high groundwater level

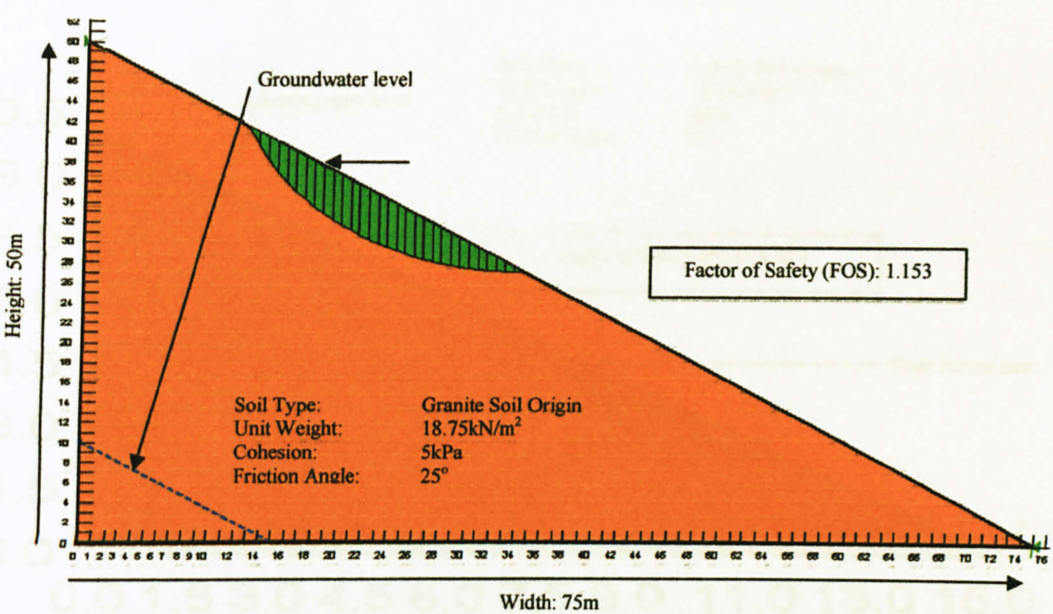


Figure4.11. Analysis of slope stability with height 50m and 75m width by factor of safety using SLOPE/W software with low groundwater level and induced of pumping method

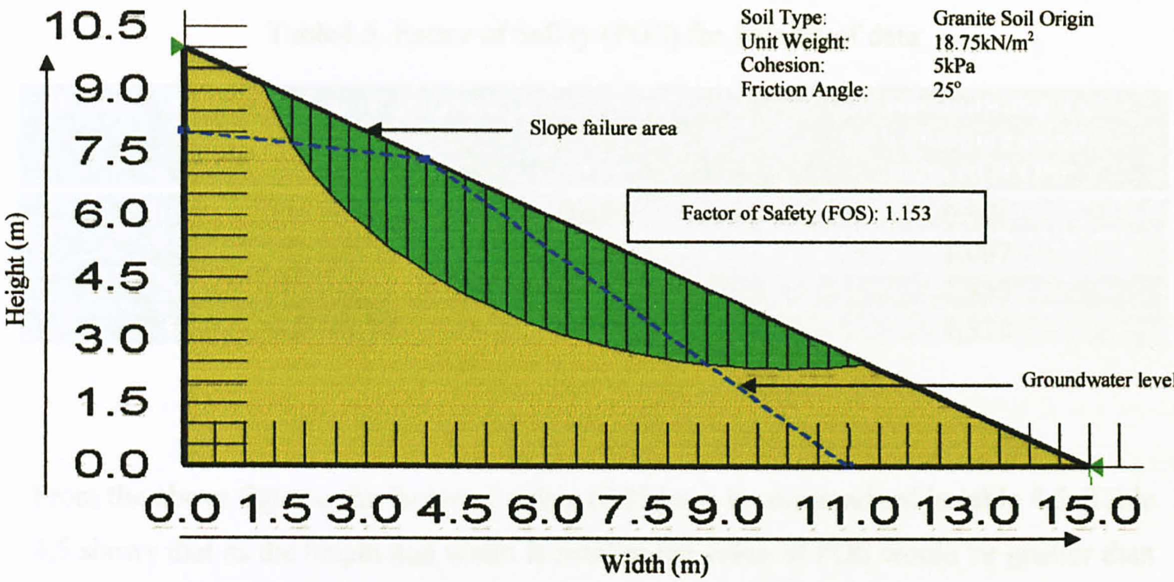


Figure4.12. Analysis of slope stability with height 50m and 75m width by factor of safety using SLOPE/W software with high groundwater level

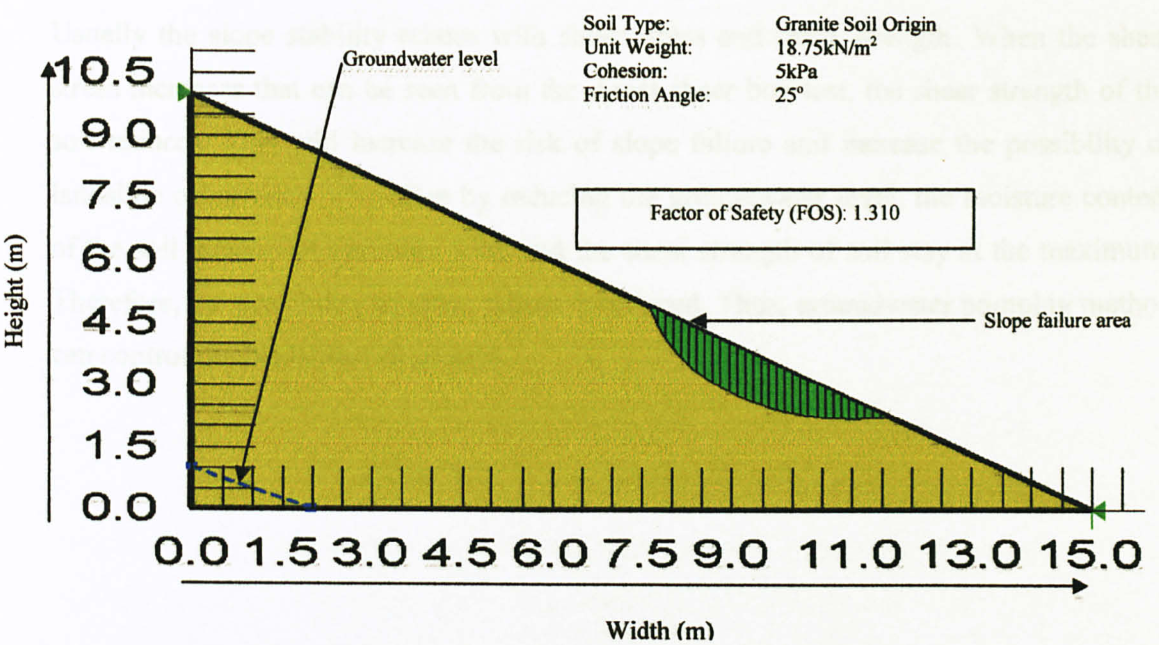


Figure4.13. Analysis of slope stability with height 50m and 75m width by factor of safety using SLOPE/W software with low groundwater level and induced of pumping method

Table4.5. Factor of Safety (FOS) for four set of data

Height: Width of Slope (m)	High Groundwater Level (FOS)	Low Groundwater Table with pumping method induced (FOS)
100:150.0	0.659	0.971
75:112.5	0.782	1.067
50:75.0	0.798	1.153
10:15.0	1.153	1.310

From the above figures, the factor of safety (FOS) can be summarized in table 4.5. Table 4.5 shows that as the height and width is smaller the value of FOS would be greater than 1. From the theory of FOS the value must be greater than 1 for slope to be considered as safe. Therefore, the height and width of the slope should be reduced so that the FOS would be greater than 1 and have a safer slope and reduce the slope failure occurrence although having the same steepness of slope and other parameter are constant. Hence lowering the groundwater table is a best solution to reduce the occurrence of landslide.

Usually the slope stability relates with shear stress and shear strength. When the shear stress increases that can be seen from the direct shear box test, the shear strength of the soil reduces. This will increase the risk of slope failure and increase the possibility of landslide occurrence. Therefore by reducing the groundwater level, the moisture content of the soil remains at optimum level and the shear strength of soil stay at the maximum. Therefore, the possibility of slope failure is reduced. Thus, groundwater pumping method can control the landslide occurrence.

CHAPTER 5

CONCLUSION

The experimental work was conducted base on the soil in Bukit Kledang. The soil is silty clay type and has intermediate plasticity index. The parameter for shear strength is also in the range that is appropriate for the soil, which is 5kPa of cohesion and 25° of friction angle. The soil plasticity index is 14% based on plastic limit and liquid limit method that was found to be 25% and 39% respectively. The permeability experiment by falling head test indicates the result as 0.0011cm/s. All the analysis of the soils was based on the soil moisture content of 7.87%.

The results of the laboratory modeling using an artificial rainfall of 200mm over a period of 24 hours show that the top 60cm of the column is having moisture content up to 30% due to the infiltration of the sprinkler water through the soil but towards the bottom of column the moisture content of the soil is almost the same as the air dry moisture content of the sample. The soils at the bottom of the column after 6 hours, 12 hours and 24 hours period of artificial rainfall were still having low moisture content with low shear stress. Thus the soil can be considered stable.

The results of increasing groundwater level show that the moisture content of the bottom part is higher than the top. This is considered to have more dramatic impact on slope stability similar to the situation when the columns were immersed with 10cm and 20cm of water in the container for 24 hours. The pore water pressure increases the moisture content of the soil above the saturation point. It found also that soil with high groundwater level, can affect the moisture content of the soil above the water table. This consequently reduces the soil shear strength and increase the shear stress and leads to lower cohesion due to the effect of lower friction.

Therefore, the results of the laboratory modeling suggest that the groundwater pumping method i.e. by pumping at slope can reduce the groundwater level increase the shear strength of the soil until ultimately increases the factor of safety (FOS) of the slope.

The analysis from the SLOPE/W software shows that base on the data at the existing ground water level, the safety index is lower than 1. Reducing the groundwater level is important to reduce the slope failure possibility as indicated by the value of FOS. For the slope height of 10m, the reduction of groundwater level could increase FOS from1.153 to 1.310 however for the slope of greater height such as 100m; the reduction could only increase from 0.659 FOS to 0.971 FOS. Therefore, for the granite residual soil recommended slope height for FOS bigger than 1.3 is 10m or less. This value is consider safe.

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APPENDIX

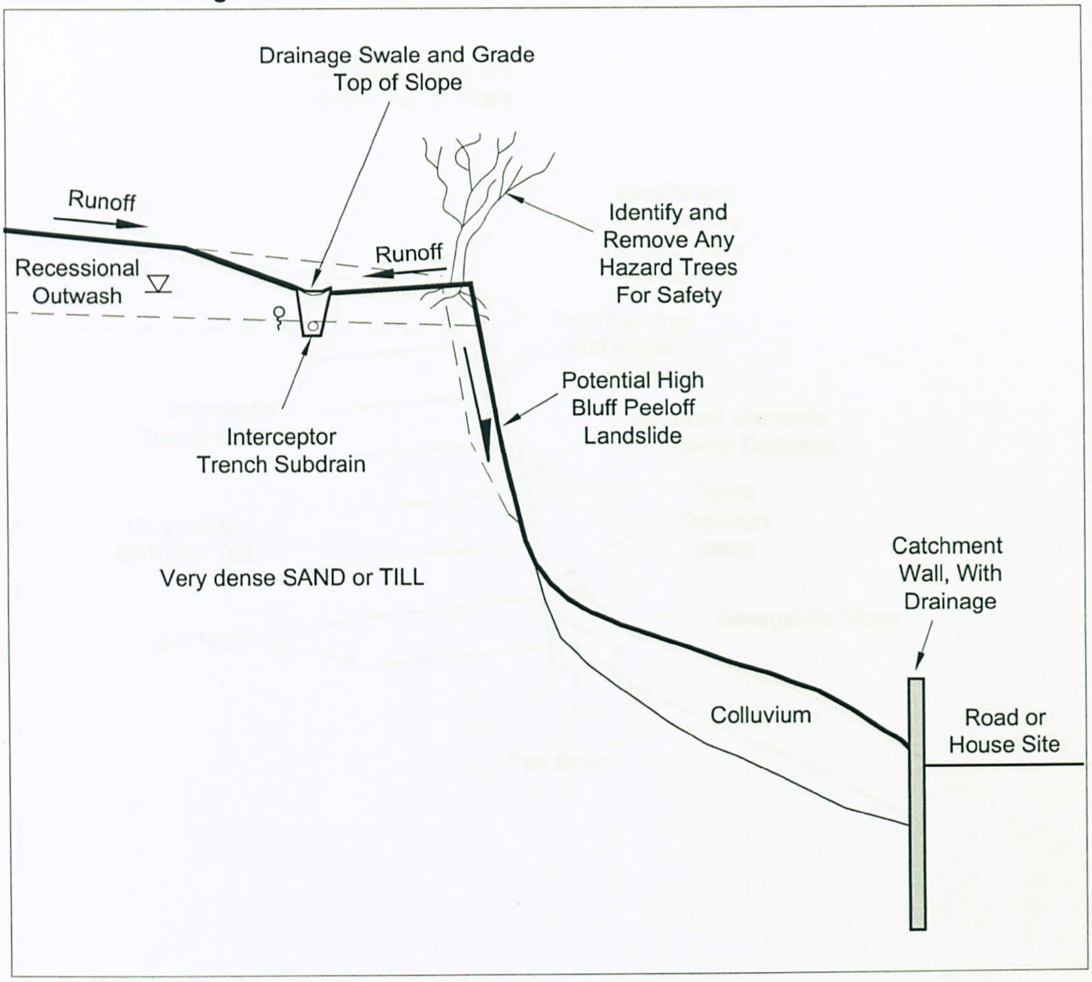
GANNT CHART FOR SECOND SEMESTER FINAL YEAR PROJECT (FYP 2)

NAME: NUR HANISAH ISMAIL
ID: 6850
TITLE: CONTROL OF LANDSLIDE BY GROUND WATER PUMPING METHOD

Legend
Process
Suggested Milestone
Required Milestone

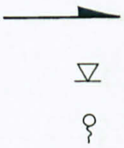
Detail / Week	1	2	3	4	5	6	7	8	9		10	11	12	13	14	15
Project Work Continue										MID SEMESTER BREAK						
> Laboratory Work																
> Practical Work																
Submission of Pre-Progress Report & Poster																
>Preparation of Poster																
> Improvise and Checking the Report																
Submission Progress Report																
Project Work Continue																
> Practical/Laboratory Work																
Submission of Pre-Dissertation																
> Improvise and Checking the Report																
Submission of Poster Presentation																
Submission of Dissertation Final Draft (SV)																
> Submission of Dissertation Final Draft (Co-Ordinator)																
Oral Presentation																
Submission of Final Dissertation (Hardbound)																

Sketch A - Drainage and Catchment



NOT TO SCALE

LEGEND



Potential Movement

Perched Water

Seepage

Seattle Landslide Study
Seattle Public Utilities
Seattle, Washington

TYPICAL HIGH BLUFF PEELOFF LANDSLIDE STABILITY IMPROVEMENTS

July 1999 W-7992-01

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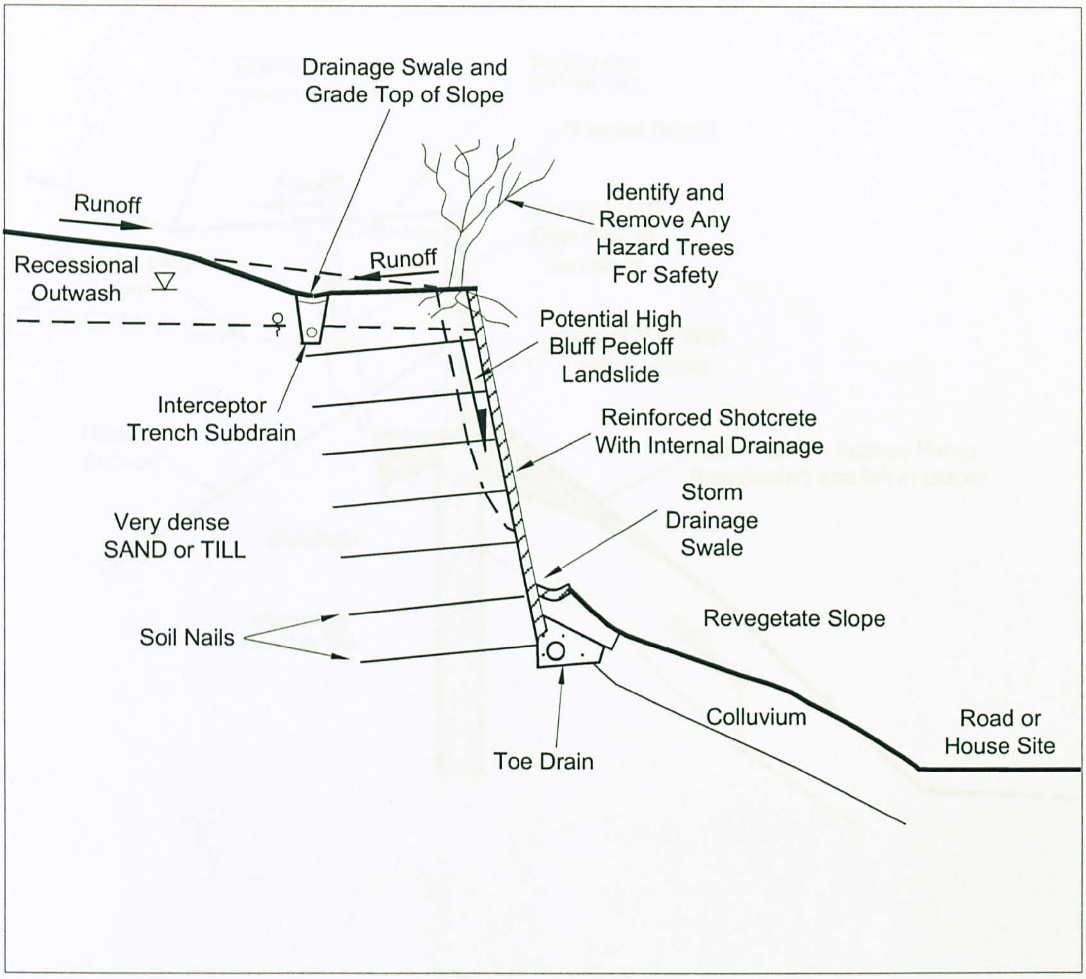
FIG. 2-1
Sheet 1 of 3

I/Documents/Mac/W Jobs/W-7900s/W-7992/01/W-7992-01 Fig. 2-1

Sketch B - Shotcrete and Soil Nails



NOT TO SCALE

LEGEND

Potential Movement

Perched Water

Seepage

Seattle Landslide Study
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TYPICAL HIGH BLUFF PEELOFF LANDSLIDE STABILITY IMPROVEMENTS

July 1999 W-7992-01

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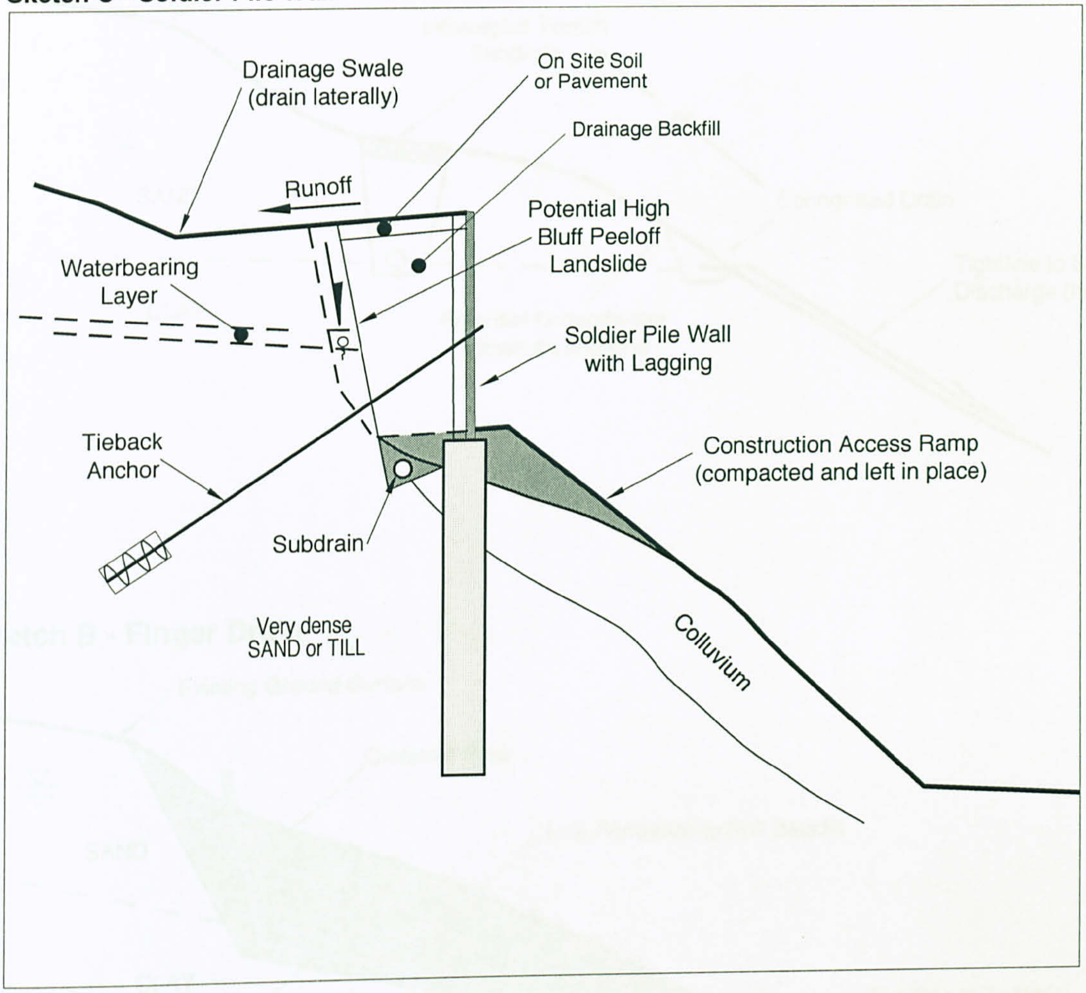
FIG. 2-1
Sheet 2 of 3

I/Documents/Mac/W Jobs/W-7900s/W-7992/01/W-7992-01 Fig. 2-1 Sheet 2 of 3

Sketch C - Soldier Pile Wall



NOT TO SCALE

LEGEND

Potential Movement

Seepage

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Seattle, Washington

TYPICAL HIGH BLUFF PEELOFF LANDSLIDE STABILITY IMPROVEMENTS

July 1999 W-7992-01

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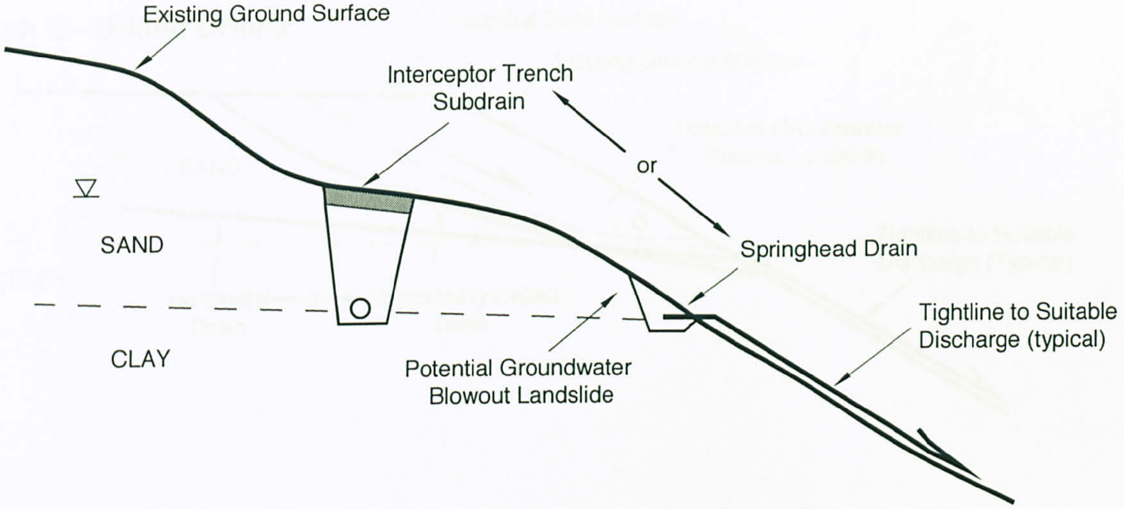
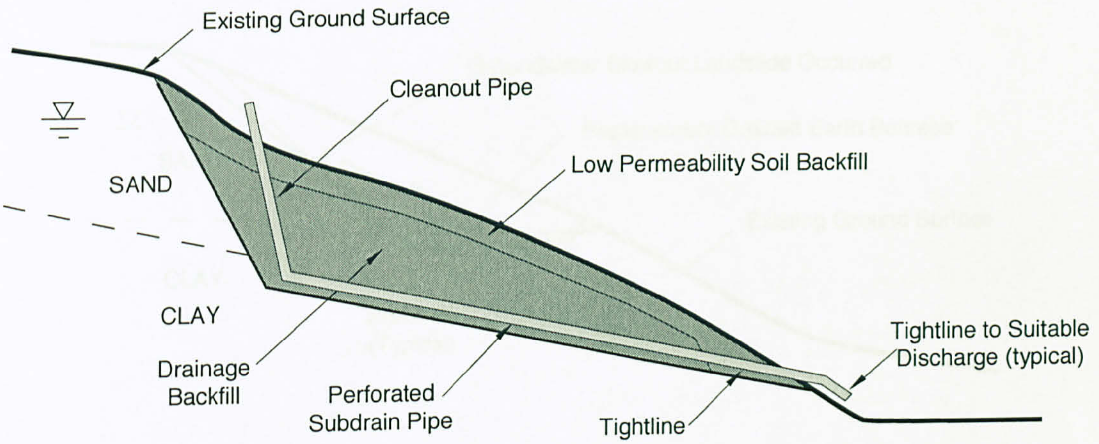
FIG. 2-1
Sheet 3 of 3

Sketch A - Interceptor Trench Subdrain and Springhead Drain



Sketch B - Finger Drain



SKETCHES NOT TO SCALE

LEGEND

▽ Perched Water

♀ Seepage

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TYPCIAL GROUNDWATER BLOWOUT LANDSLIDE STABILITY IMPROVEMENTS

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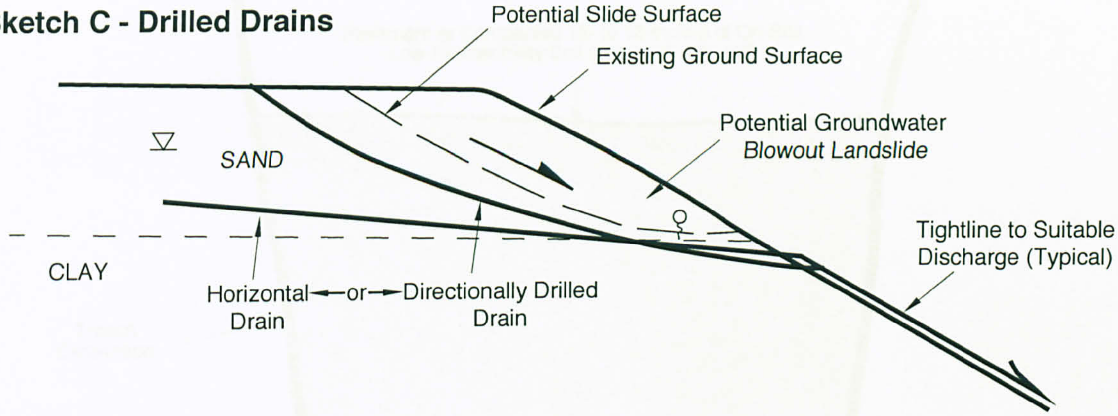
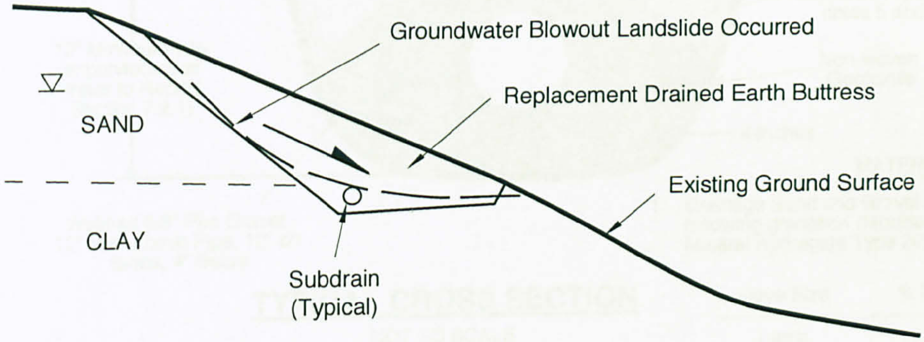
FIG. 2-2
Sheet 1 of 2

Sketch C - Drilled Drains



Sketch D - Replacement Earth Buttress



SKETCHES NOT TO SCALE

LEGEND

Potential Movement

Perched Water

Seepage

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TYPICAL GROUNDWATER BLOWOUT LANDSLIDE STABILITY IMPROVEMENTS

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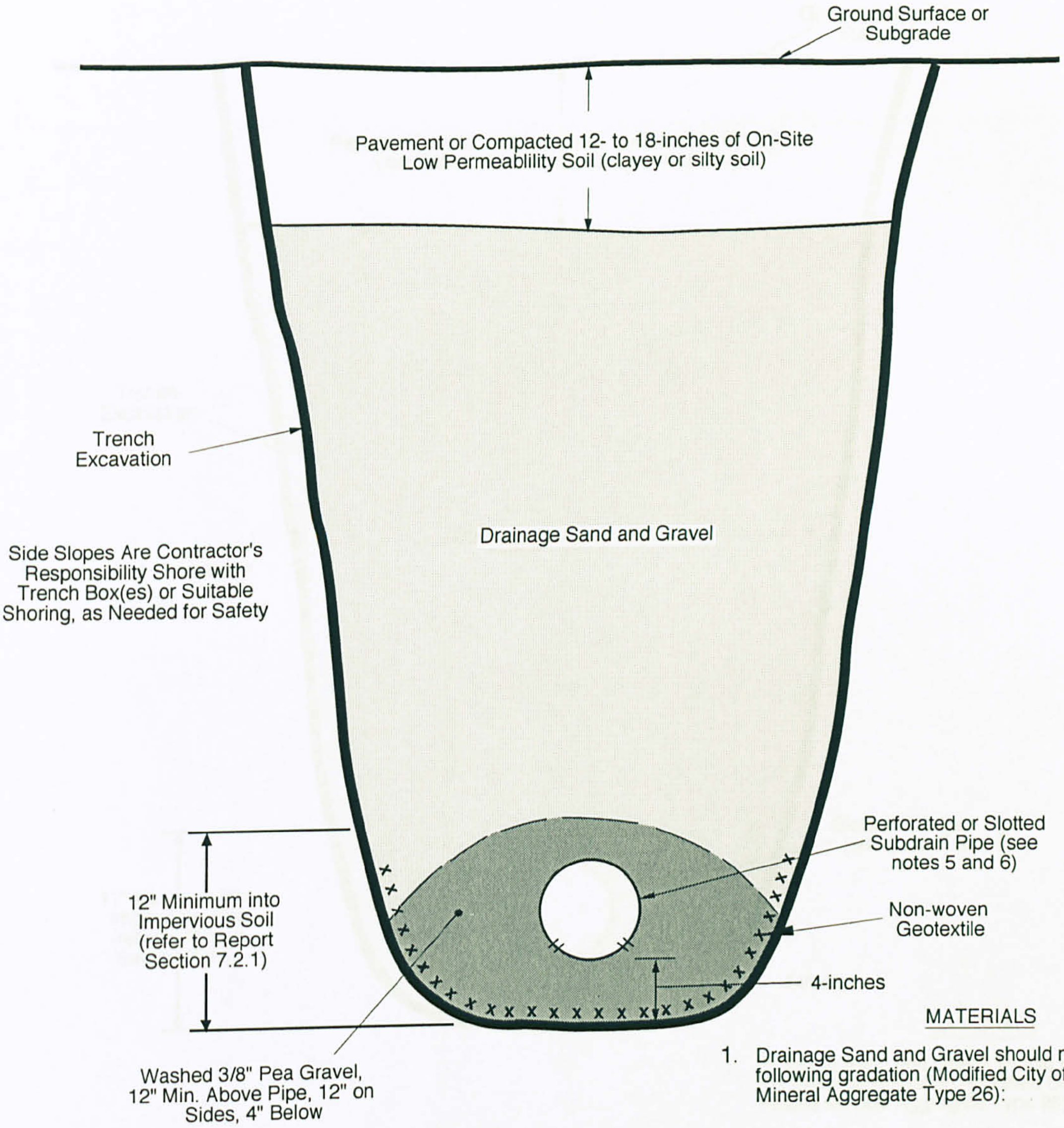
FIG. 2-2
Sheet 2 of 2



TYPICAL CROSS SECTION

NOT TO SCALE

NOTES

1. This figure is not for construction. It should only be used for information pertaining to potential design concepts. Final design should be based on site-specific conditions and accomplished by a geotechnical engineer licensed as a professional engineer.
2. Possible caving soil conditions may require that the subdrain pipe and backfill be placed concurrently with the trench excavation.
3. Extend pipe by means of a tightline to a suitable discharge point. Where subdrain pipe changes to a tightline, provide impervious dam (concrete or clay) so as to force all water into the tightline (see Figure 2-8).
4. Drain backfill should be compacted to a relatively dense condition (see Report Section 7.2.1).
5. Perforated or slotted subdrain pipe; tight joints; sloped to drain (6"/100' min. slope); provide clean-outs; min. diameter: 6 inches.
6. Perforated pipe holes (1/8-in. to 3/8-in. dia.) to be in lower half of pipe with lower quarter segment unperforated for water flow. Slotted pipe to have 1/8" maximum slot width.

MATERIALS

1. Drainage Sand and Gravel should meet the following gradation (Modified City of Seattle Mineral Aggregate Type 26):

Sieve Size	% Passing by Weight
1-inch	100
3/4-inch	85 to 95
1/4-inch	30 to 60
No. 8	20 to 50
No. 50	3 to 12
No. 200	0 to 1
(by wet sieving)	(non-plastic fines)

An alternative to drainage sand and gravel is City of Seattle Mineral Aggregate Type 6 (washed sand).

2. Washed 3/8" pea gravel to meet City of Seattle Mineral Aggregate Type 9.

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TYPICAL TRENCH SUBDRAIN INTERCEPTOR TRENCH AND FINGER DRAIN

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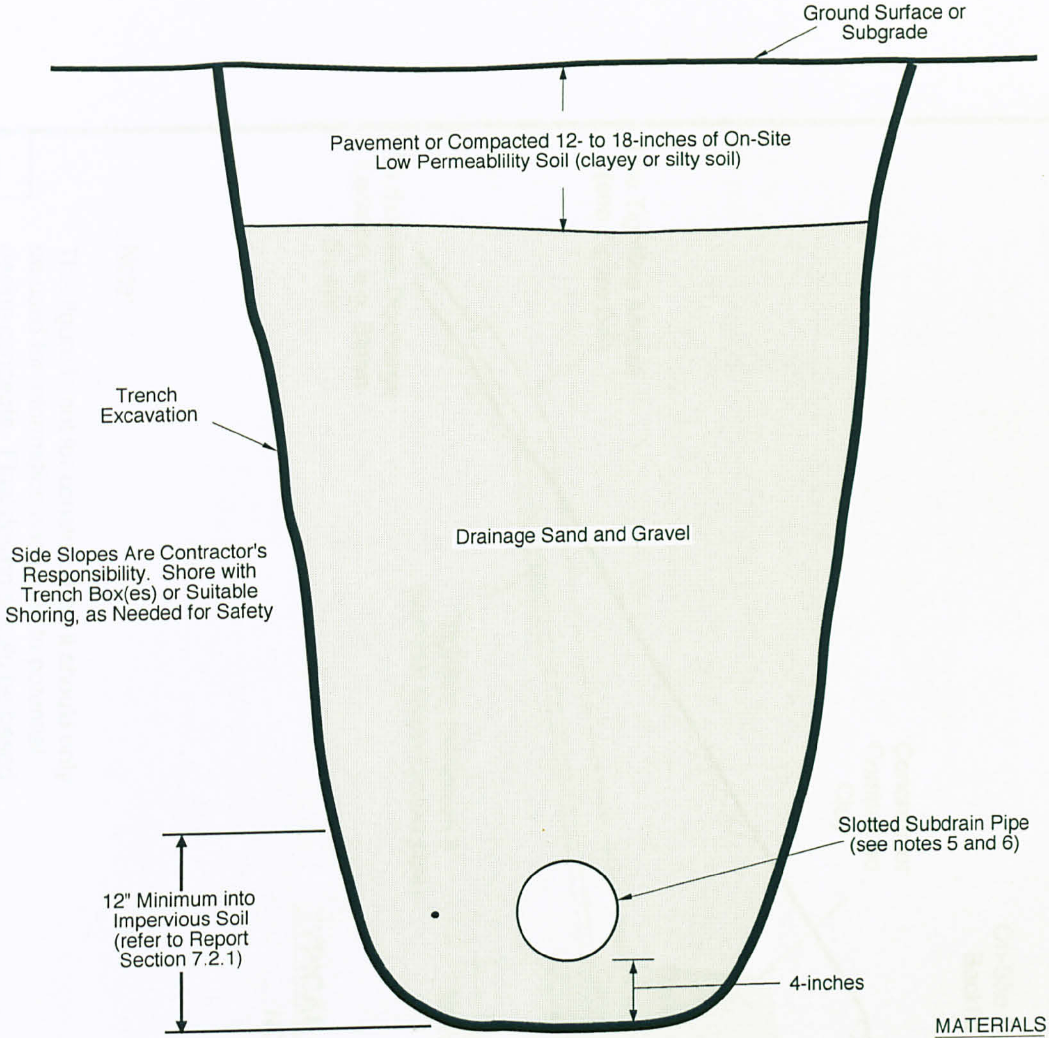
FIG. 2-7
Sheet 1 of 2



TYPICAL CROSS SECTION

NOT TO SCALE

NOTES

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2. Possible caving soil conditions may require that the subdrain pipe and backfill be placed concurrently with the trench excavation.
3. Extend pipe by means of a tightline to a suitable discharge point. Where subdrain pipe changes to a tightline, provide impervious dam (concrete or clay) so as to force all water into the tightline (see Figure 2-8).
4. Drain backfill should be compacted to a relatively dense condition (see Report Section 7.2.1).
5. Slotted subdrain pipe; tight joints; sloped to drain (6"/100' min. slope); provide clean-outs; min. diameter: 6 inches.
6. Slotted pipe to have 1/8" maximum slot width.

MATERIALS

1. Drainage Sand and Gravel should meet the following gradation (Modified City of Seattle Mineral Aggregate Type 26):

Sieve Size	% Passing by Weight
1-inch	100
3/4-inch	85 to 95
1/4-inch	30 to 60
No. 8	20 to 50
No. 50	3 to 12
No. 200	0 to 1
(by wet sieving)	(non-plastic fines)

An alternative to drainage sand and gravel is a 50-50 mixture of washed pea gravel (Mineral Aggregate Type 9) and washed sand (mineral aggregate type 6).

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Seattle, Washington

TYPICAL TRENCH SUBDRAIN INTERCEPTOR TRENCH AND FINGER DRAIN

October 1998 W-7992-01

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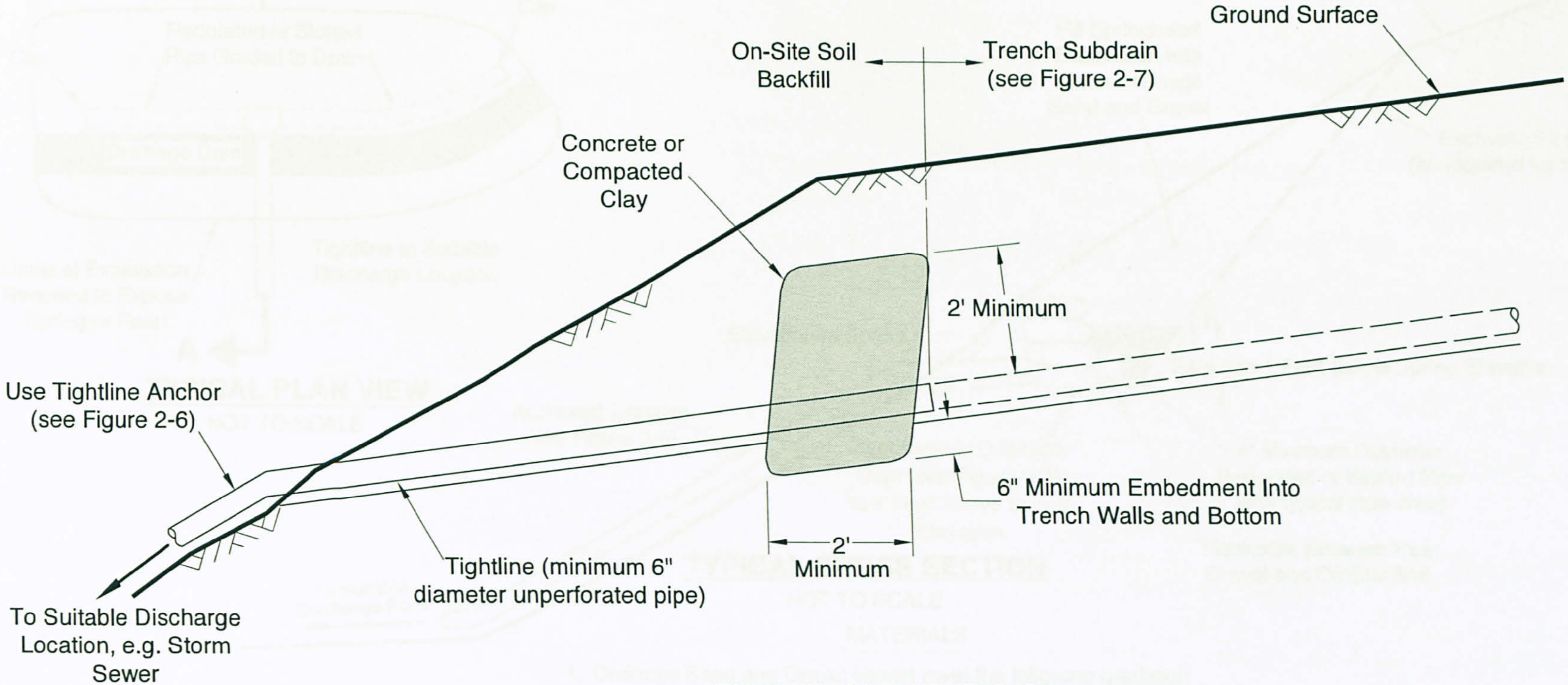
FIG. 2-7
Sheet 2 of 2



TYPICAL CROSS SECTION

NOT TO SCALE

Note:

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FIG. 2-8

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TYPICAL DRAINAGE DAM DETAIL

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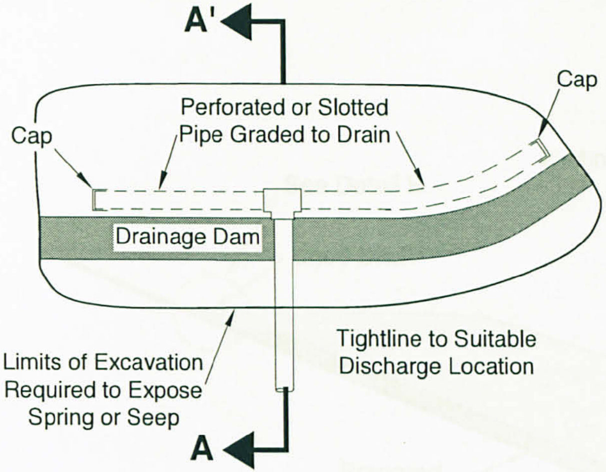
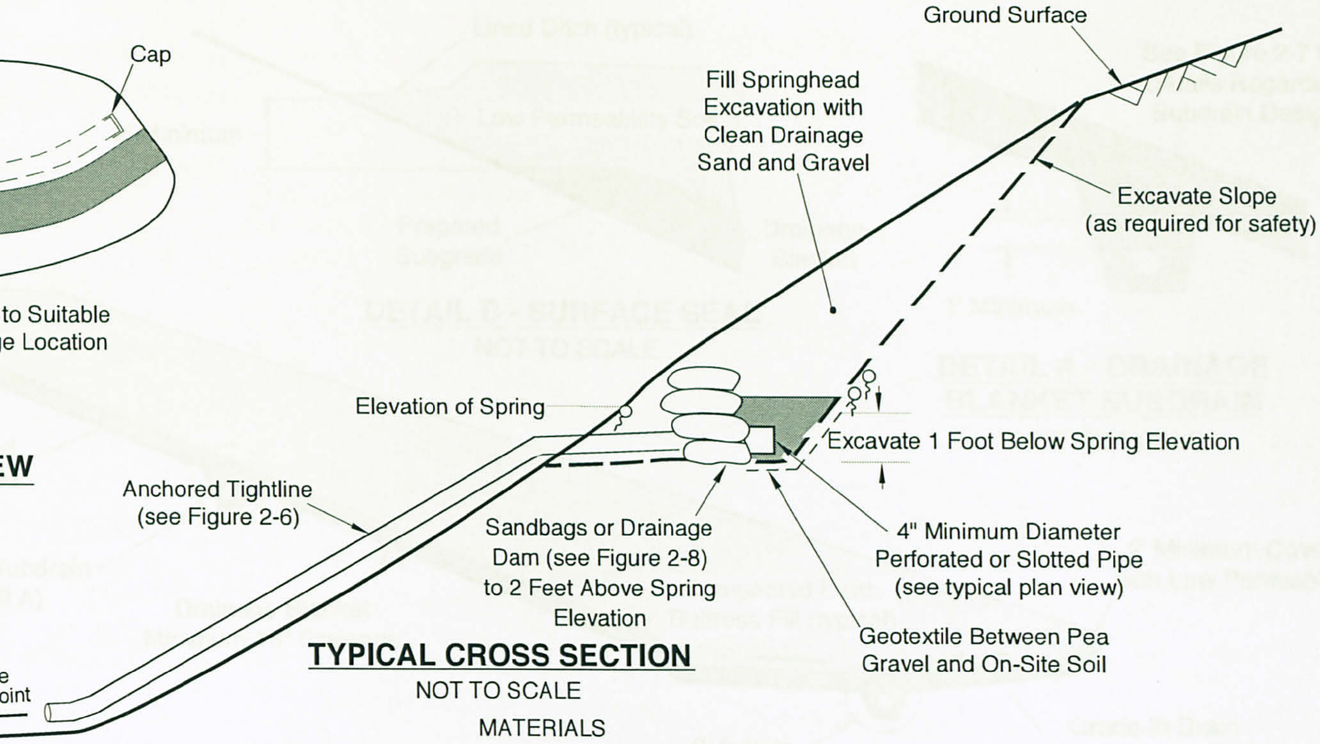
FIG. 2-8



TYPICAL PLAN VIEW

NOT TO SCALE



TYPICAL CROSS SECTION

NOT TO SCALE

MATERIALS

1. Drainage Sand and Gravel should meet the following gradation (Modified City of Seattle Mineral Aggregate Type 26):

Sieve Size	% Passing by Weight
1-inch	100
3/4-inch	85 to 95
1/4-inch	30 to 60
No. 8	20 to 50
No. 50	3 to 12
No. 200	0 to 1
(by wet sieving)	(non-plastic fines)

An alternative drainage sand and gravel is a 50-50 mixture of washed pea gravel and washed concrete sand.

2. Washed 3/8" Pea Gravel to Meet City of Seattle Mineral Aggregate Type 9.

NOTES

1. This figure is not for construction. It should only be used for information pertaining to potential design concepts. Final design should be based on site-specific conditions and accomplished by a geotechnical engineer licensed as a professional engineer.

2. Perforated pipe holes (1/8-inch to 3/8-inch diameter) to be in lower half of pipe with lower quarter segment unperforated for water flow. Slotted pipe to have 1/8" maximum slot width.

FIG. 2-9

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TYPICAL SPRINGHEAD DRAIN DETAIL

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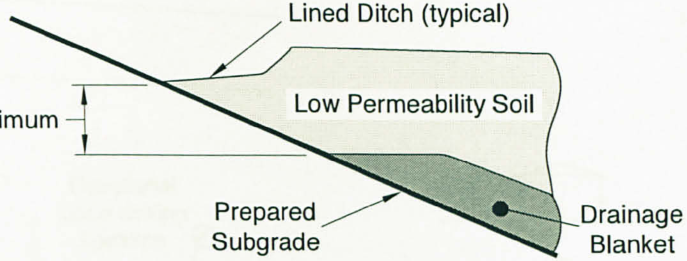
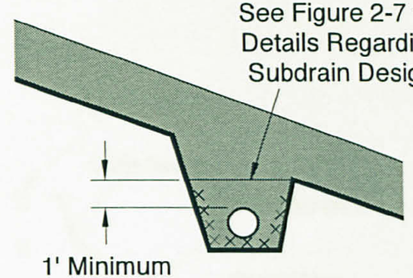
FIG. 2-9



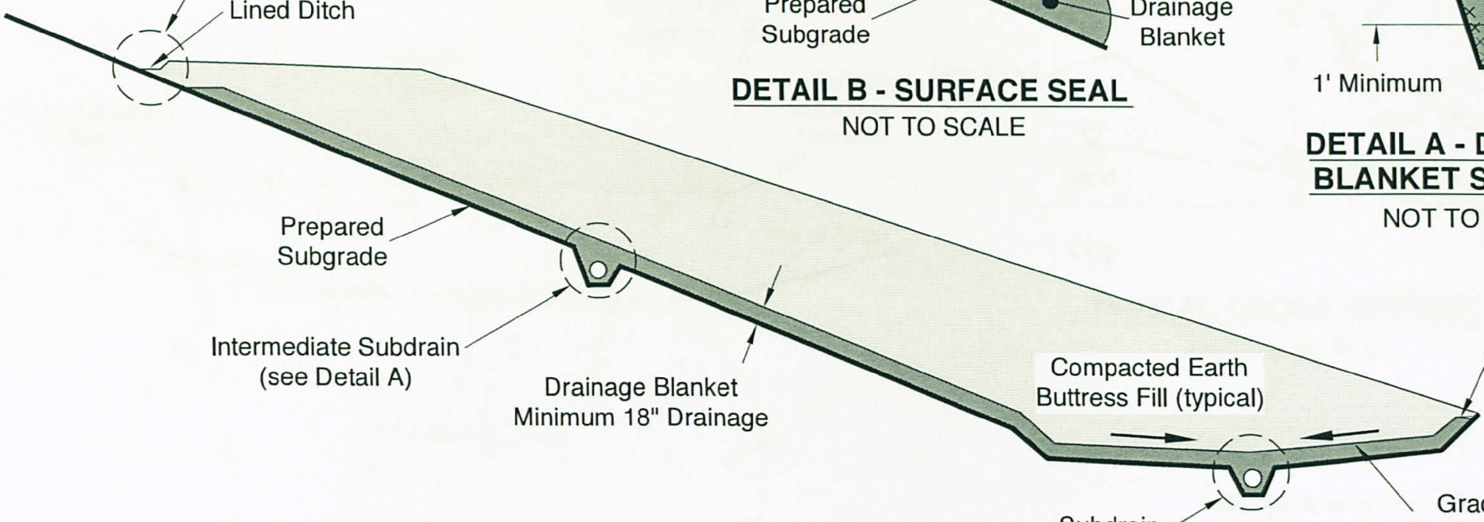
DETAIL B - SURFACE SEAL

NOT TO SCALE



DETAIL A - DRAINAGE BLANKET SUBDRAIN

NOT TO SCALE



DRAINAGE BLANKET DETAIL

NOT TO SCALE

MATERIALS

Drainage Sand and Gravel should meet the following gradation (Modified City of Seattle Mineral Aggregate Type 26):

Sieve Size	% Passing by Weight
1-inch	100
3/4-inch	85 to 95
1/4-inch	30 to 60
No. 8	20 to 50
No. 50	3 to 12
No. 200	0 to 1
(by wet sieving)	(non-plastic fines)

An alternative drainage sand and gravel is a 50-50 mixture of washed pea gravel and washed concrete sand.

Note:

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TYPICAL DRAINAGE BLANKET DETAIL

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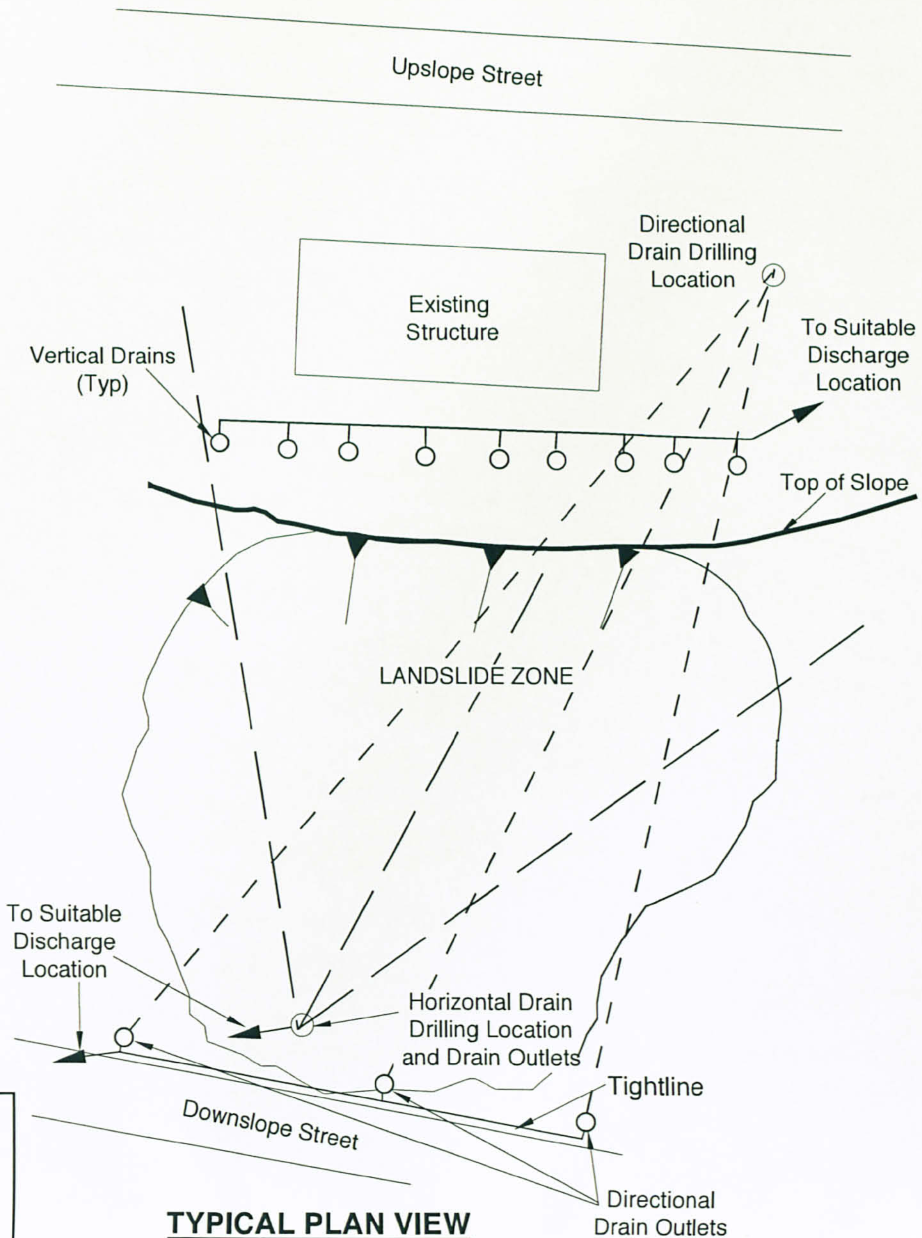
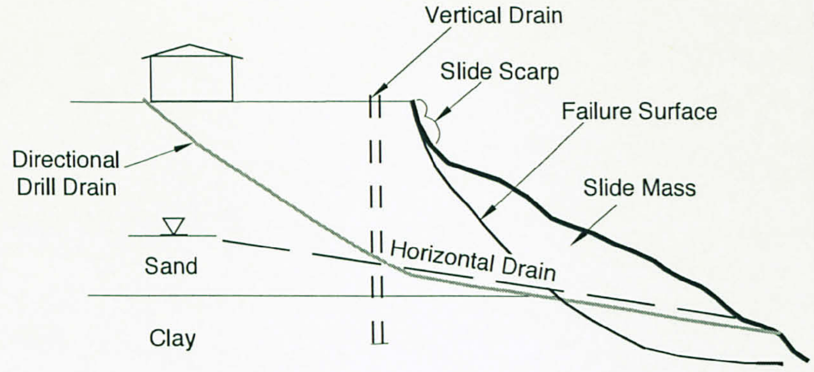
FIG. 2-10

FIG. 2-10



TYPICAL PLAN VIEW
NOT TO SCALE



TYPICAL CROSS SECTION
NOT TO SCALE

Note:

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TYPICAL HORIZONTAL, DIRECTIONAL AND VERTICAL DRAIN SKETCH

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FIG. 2-11

FIG. 2-11